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August 1994

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Conemaugh River Lake Sediment Removal Study

by Roy Wade, Gary E. Freeman,
Allen M. Teeter, William A. Thomas

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Prepared for U.S. Army Engineer District, Pittsburgh

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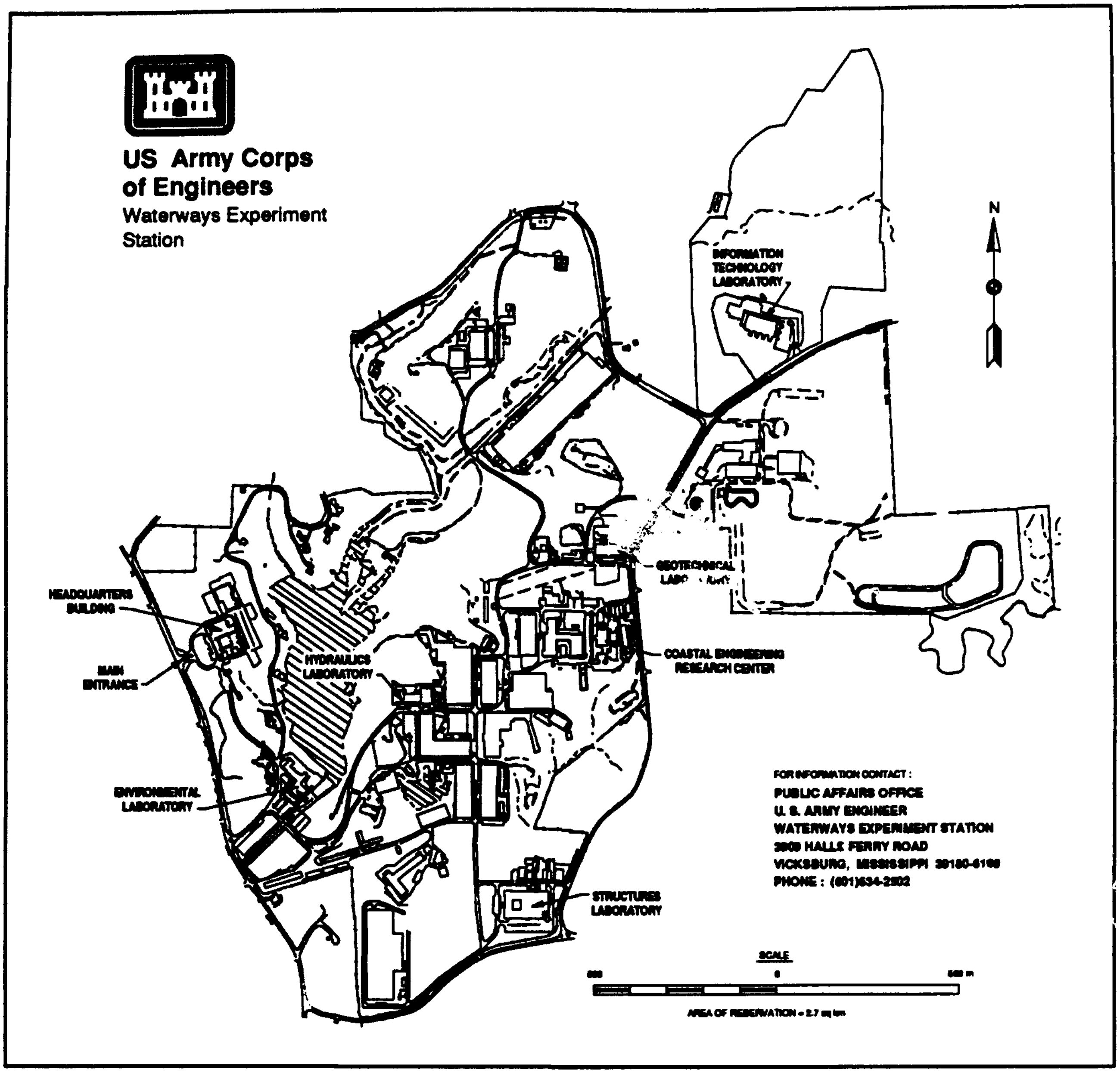

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Preface

This study was conducted as a part of the Conemaugh River Lake Sediment Removal Study. This report was prepared at the U.S. Army Engineer Waterways Experiment Station (WES) in cooperation with the U.S. Army Engineer District, Pittsburgh.

Project manager for the Pittsburgh District was Mr. Marshall Fausold. Project manager for WES was Mr Roy Wade. The settling and modified elutriate studies were conducted between March 1990 and April 1990 in the WES Environmental Laboratory (EL). The numerical model was developed during the period July 1990 to August 1992 in the WES Hydraulics Laboratory (HL).

The settling and modified elutriate tests portion of this report was written by Mr. Wade, Environmental Restoration Branch (ERB), Environmental Engineering Division (EED), EL, WES. Laboratory support was provided by Messrs. Fred Ragan and Mike Channell both of ERB and Mrs. Linda Mayfield of the WES Structures Laboratory. This portion of the study was conducted under the direct supervision of Mr. Norman R. Francingues, Jr., Chief, ERB, and under the general supervision of Dr. Raymond L. Montgomery, Chief, EED, and Dr. John W. Keeley, Director, EL.

The numerical model portion of the report was written by Dr. Gary E. Freeman and Mr. William A. Thomas, Math Modeling Branch, Waterways Division, and Mr. Allen M. Teeter, Estuarine Branch, Estuaries Division, HL, WES. This portion of the study was conducted under the direct supervision of Messrs. Frank A. Herrmann, Jr., Director, HL, and R. A. Sager, Assistant Director, HL.

At the time of publication of this report, Dr. Robert W. Whalin was Director of WES. COL Bruce K. Howard, EN, was Commander.

This report should be cited as follows:

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Conversion Factors, Non-SI to SI Units of Measurement

Non-SI units of measurement used in this report can be converted to SI units as follows:

Multiply	By	To Obtain
acres	4,046.873	square meters
acre-feet	1,233.489	cubic meters
cubic feet per second	0.0283165	cubic meters per second
cubic yards	0.7645549	cubic meters
Fahrenheit degrees	5/9	Celsius degrees or kelvins[1]
feet	0.3048	meters
gallons (U.S. liquid)	3.785412	cubic decimeters
inches	2.54	centimeters
miles (U.S. statute)	1.609347	kilometers
pounds (mass)	0.4535924	kilograms
pounds per cubic feet	16.01846	kilograms per cubic meter
square feet	0.09290304	square meters
square inches	6.4516	square centimeters
square miles (U.S. statute)	2589.998	square kilometers
tons (short, 2000 lb)	907.1847	kilograms
yards	0.9144	meters

[1] To obtain Celsius (C) temperature readings from Fahrenheit (F) readings, use the following formula: $C = (5/9)(F - 32)$. To obtain kelvin (K) readings, use $K = (5/9)(F - 32) + 273.15$.

1 Introduction

Conemaugh River Lake

Conemaugh River Lake is a flood-control project in southwestern Pennsylvania on the Conemaugh River upstream from the confluence of Loyalhanna Creek which forms the Kiskiminetas River, a tributary of the Allegheny River (Figure 1). The upstream basin of the Conemaugh River contains about 1,351 square miles.[1] The topography of the basin is characterized by high, rugged, rolling hills in the lower reaches and higher, deeply dissected, mountainous areas in the upper reaches. The lake is downstream from Johnstown, PA, the site of infamous Johnstown Flood in 1889. The lake serves primarily as a flood-control reservoir (U.S. Army Engineer District, Pittsburgh 1985).

Conemaugh River Lake was formed by the closure of the Conemaugh River Dam in late 1952. The closure of the dam created a pool that impounds water on the Conemaugh River and on Blacklick Creek, a major tributary (Figure 2). The Conemaugh River Lake at maximum elevation extends over Two Lick Creek and Blacklick Creek. Normal operating pool has been gradually raised over the past 40 years from 880 to 890 ft prior to the hydropower plant installation in the late 1980s. Normal pool has been about 900 ft National Geodetic Vertical Datum (NGVD) since the hydropower plant was installed.

The Problem

Sedimentation surveys of the lake conducted in 1966 and 1982 indicated an accumulation of sediment resulting in a reduction in storage capacity of 11,342 acre-ft. This represents a reduction in gross reservoir storage capacity of 4.14 percent or an annual average storage loss of 0.14 percent over the study period. Much of the accumulated sediment is located immediately upstream of the dam. The sediment, deposited since construction of the dam, exceeds 30 ft in thickness in the lower 3 miles of the reservoir except for a

[1] A table of factors for converting non-SI units to SI units is given on page viii.

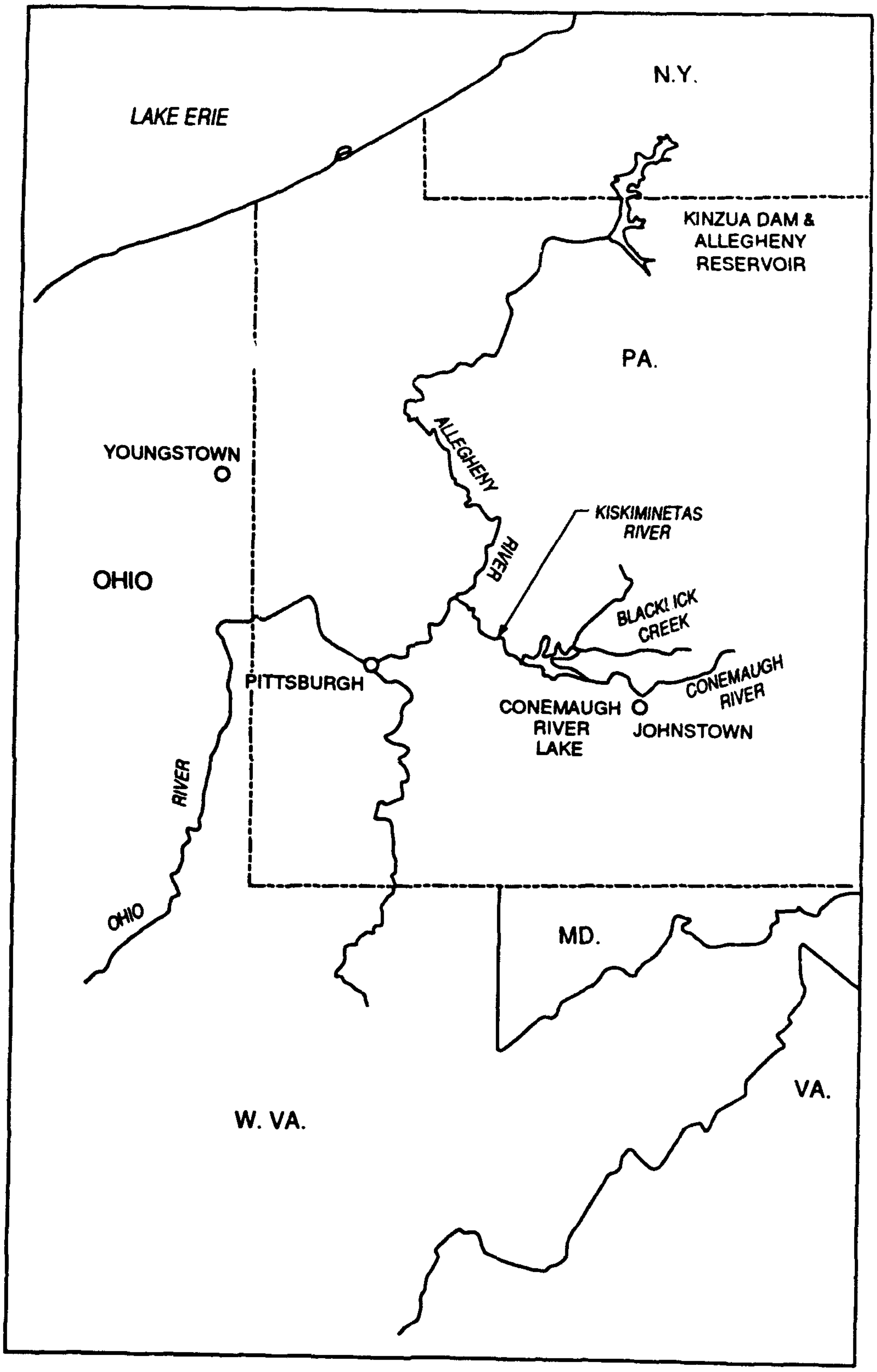

Figure 1. Vicinity map

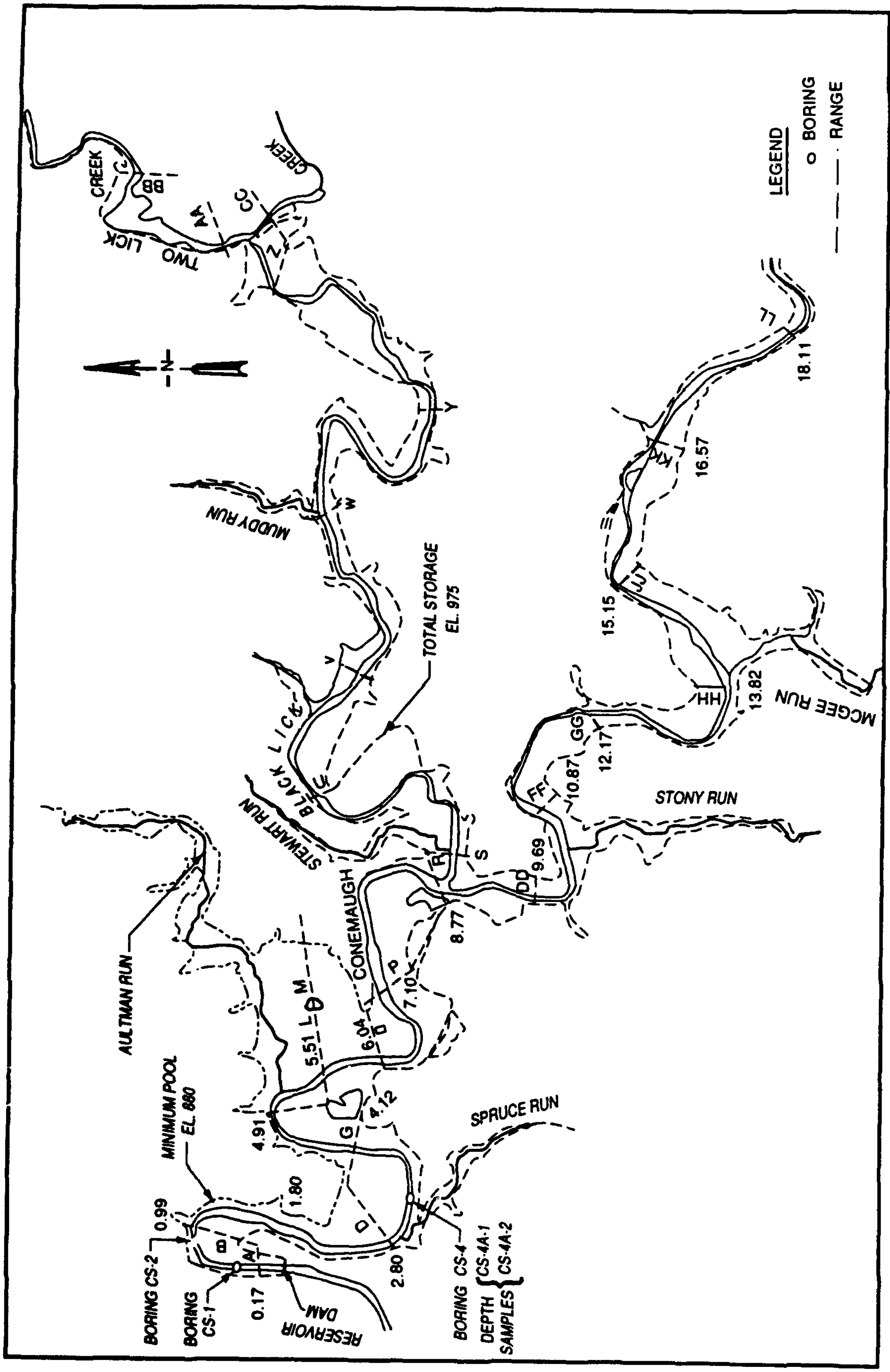

Figure 2. Study area map

low flow channel near Blacklick Creek (Figure 3). The deposited sediment is interfering with the operation of the outlet structure conduits.

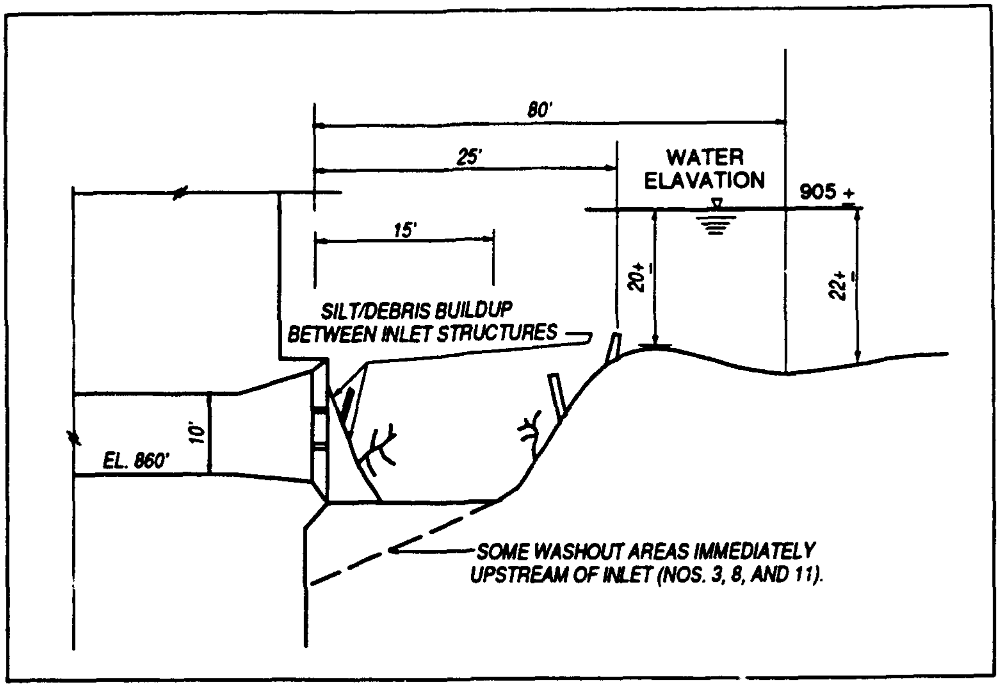

Figure 3. Schematic showing deposition pattern at dam

Organization of Report

This report is presented in four chapters. The introduction and background information on the Conemaugh River Lake is discussed in Chapter 1. Chapter 2 presents methodologies and results of settling and modified elutriate tests. The development and evaluation of a numerical model are discussed in Chapter 3. Finally, conclusions and recommendations are presented in Chapter 4. Appendix A includes detailed test results.

2 Settling and Modified Elutriate Tests

Background

Sediment removal to restore flexibility of operation of the conduits at Conemaugh River Lake Dam is required. One alternative being considered for the Conemaugh River Lake is hydraulic dredging with temporary or permanent disposal of the dredged material in an upland confined disposal facility (CDF). The conceptual design of the CDF requires an evaluation of the settling behavior and properties of the dredged material to be placed therein in order to estimate storage requirements and to promote good settling within the CDF. Efficient solids removal benefits CDF effluent quality by reducing possible particulate-associated contaminants along with lowering suspended solids concentrations.

Settling tests were conducted in the Environmental Laboratory (EL) at the U.S. Army Engineer Waterways Experiment Station (WES). Settling test procedures reported by Montgomery (1978), Palermo, Montgomery, and Poindexter (1978), and Palermo and Thackston (1988) were used to predict the concentration of suspended solids in the effluent for given operational conditions at the Conemaugh River Lake site. Modified elutriate tests as reported by Thackston and Palermo (1990) and Palermo (1984) were also conducted to predict both the dissolved concentrations of contaminants in milligrams per liter and particle-associated contaminant fractions of the suspended solids in milligrams per kilogram of suspended solids under quiescent settling conditions. Using the column settling test results, WES researchers determined the storage capacity of a CDF based on effluent suspended solids concentration. Using results from both the column settling test and the modified elutriate test, researchers predicted the total concentration of contaminants in the effluent.

A Typical CDF

A CDF is a diked enclosure constructed to retain dredged material placed in the site. To be effective, the CDF must be designed to provide adequate storage capacity for the settled sediments and efficient sedimentation to minimize the discharge of suspended solids (Montgomery, Thackston, and Parker 1983). Figure 4 shows an active CDF where the dredged material undergoes sedimentation, resulting in a "thickened" deposit of settled material overlain by the clarified supernatant. The supernatant waters are normally discharged from the site as effluent, which may contain dissolved and/or particulate-associated contaminants.

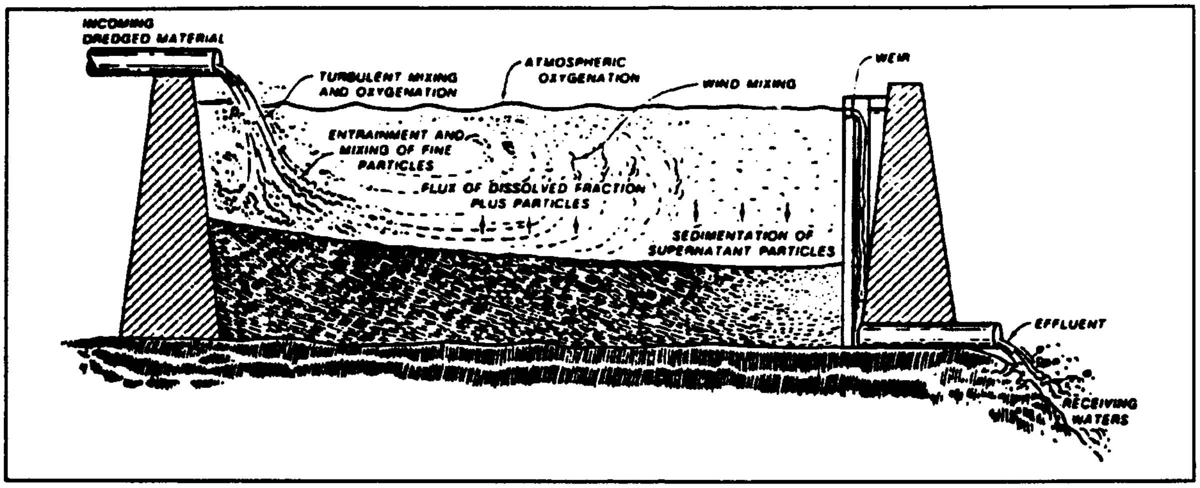

Figure 4. Schematic of an active CDF

Figure 4 also shows several factors influencing the concentration of suspended particles and contaminants present in supernatant waters. As dredged material slurry enters the ponded water, finer particles remain suspended in the water column at the point of entry due to turbulence and mixing. The suspended particles are partially removed from the water column by gravity settling. Some of the settled particles may reenter the water column because of the upward flow of water through the slurry mass during thickening and may reenter the water column by wind and/or surface wave action. If supernatant water is released during active phases of disposal, all solids cannot be retained. Therefore, dissolved and particulate-associated contaminants may be transported with the particles in the effluent to the receiving water outside the containment area.

Purpose

The purpose of this section is to document and present the results of the column settling and modified elutriate tests performed as part of the sediment removal study and to apply the results to conceptual design of a CDF.

Testing Objectives

The objective of the settling tests was to predict the settling behavior of Conemaugh River Lake composite sediment when hydraulically dredged and placed in a CDF. The objective of the modified elutriate test was to predict the quality of the effluent by accounting for the dissolved concentrations of contaminants and the solid contaminant fraction associated with the total suspended solids released.

Prior to running the settling and modified elutriate tests, homogenized sediment samples were collected and analyzed for organic constituents, inorganic constituents, and radionuclides. Historical data have not shown evidence of any significant levels of contamination in the sediments requiring removal at the Conemaugh River Lake. However, it is not uncommon for dredged material resulting from the sedimentation in rivers near industrialized areas to contain contaminants. Mining, lumbering, and farming operations, and the outfalls from factories and city wastewater treatment systems may result in contaminant levels in the sediment that may be high enough to cause concern during dredging and disposal operations.

Scope Of Work

The scope of work included performing laboratory column settling tests on Conemaugh River Lake sediment and estimating the volume requirements for solid storage and suspended solids removal effectiveness for the CDF. An initial screening for contamination was performed to determine if there was a reason to believe that the sediment contained any contaminant at a significant concentration and to identify the contaminants that should be analyzed in the modified elutriate test. For each contaminant of concern, the modified elutriate test procedure was run to define the dissolved concentration and the fraction of the particle-associated contaminant in the total suspended solids under quiescent settling conditions. This procedure also accounts for geochemical changes occurring in the disposal area during active disposal operations.

Experimental Procedures

General

This part of the report describes laboratory testing conducted to predict solids storage capacity and effluent quality of the proposed CDF. Samples of sediment and water were collected and used to conduct the column settling and modified elutriate tests. Results from both of these tests were used to predict the total concentration of contaminants that may be present in the effluent. A flowchart illustrating the effluent quality prediction technique is shown in Figure 5.

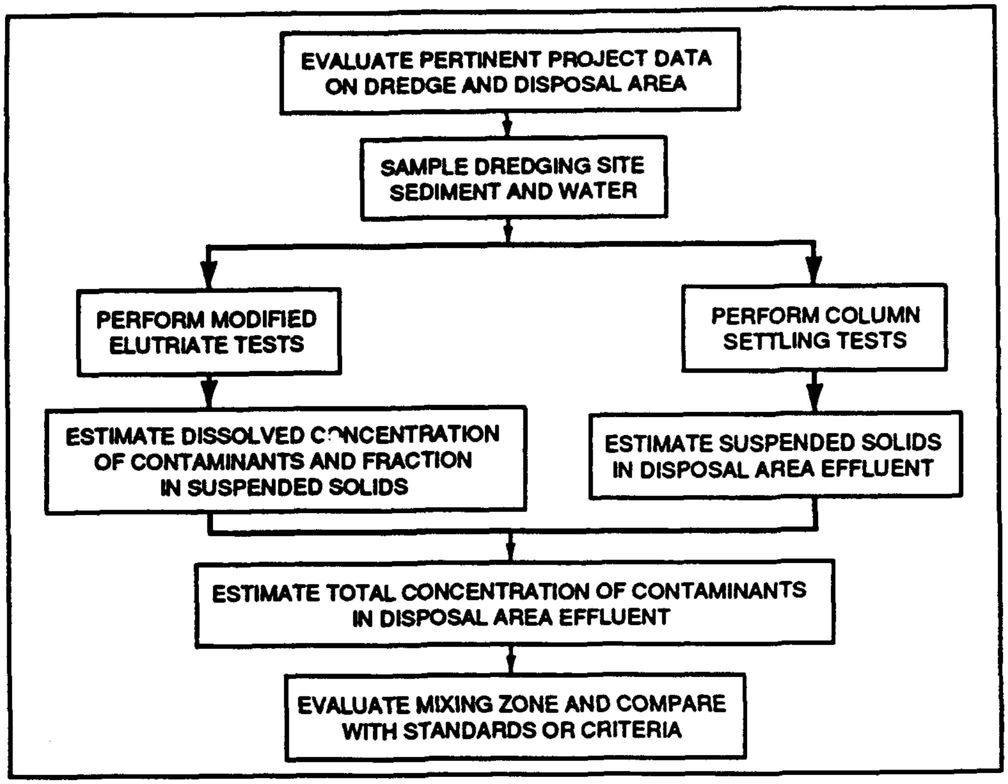

Figure 5. Steps for predicting effluent water quality (Palermo 1984)

Sample collection

Samples of bottom sediment and site water were collected from the Conemaugh River Lake during September 1989 by the Pittsburgh District. The sediment and site water were shipped to WES in 4-in. polyvinyl (PVC) pipes and a 35-gal drum, respectively. The WES Hydraulics Lab (HL) used eight of the borings for particle size gradation and specific gravity determination. The WES EL composited and homogenized the remaining samples in a 55-gal drum. The sample was homogenized with a Lightnin mixer for 1 hr. Total solids were run to ensure a homogenized sample.

Settling tests

The settling tests followed procedures found in WES Technical Report DS-78-10 (Palermo, Montgomery, and Poindexter 1978), EM 1110-2-5027 (OCE 1987), and WES Technical Report D-88-9 (Palermo and Thackston 1988).

The tests involve mixing sediment and site water to simulate a dredged material slurry, placing the material in a settling column, and observing each of several types of settling (i.e., discrete, zone, flocculent, and compression) behavior. The general procedures are described below.

Procedures. The flocculent settling test consisted of measuring the concentration of suspended solids at various depths and time intervals in a settling column. An interface formed near the top of the settling column during the first day of the test; therefore, sedimentation of the material below the interface is described by zone settling. The flocculent test procedure was continued only for that portion of the water column above the interface. Samples of the settling slurry were extracted from each sampling port above the liquid-solid interface at different time intervals. The suspended solids concentrations of the extracted samples were determined. Substantial reductions of suspended solids are expected to occur during the early part of the test, but reductions should lessen at longer retention times (EM 1110-2-5027).

The zone settling test consisted of placing a slurry in a sedimentation column and reading and recording the fall of the liquid-solids interface with time. These data are plotted as depth from the surface to the interface versus time. The slope of the constant velocity settling zone of the curve is the zone settling velocity, which is a function of the initial slurry concentration.

A compression settling test must be run to obtain data for estimating the volume required for initial storage of the dredged material. For slurries exhibiting zone settling, the compression settling data can be obtained by continuing the zone settling test for a period of 15 days, so that a relationship of log of concentration versus log of time in the compression settling range is obtained (EM 1110-2-5027).

Slurry preparation. The target slurry concentration selected for the settling tests was 150 g/L, the suggested default value for hydraulically dredged slurry since the dredged material actual influent concentration was not known. The slurry was prepared by mixing the Conemaugh River Lake composite sediment with water collected from the site. To achieve the target slurry concentration for the composite material, approximately 6 gal of sediment, which had an average solids concentration of 476 g/L, was mixed with 14 gal of site water using a Lightnin mixer. The slurry was pumped from a 55-gal drum with a positive displacement pump into an 8-in. diam, 7-ft column, with ports at 0.5-ft intervals starting at the 7.0-ft depth (see Figure 6). After the slurry was thoroughly mixed and pumped into the column, six samples for total solids were extracted from ports at the 6.0-, 5.0-, 4.0-, 3.0-, 2.0-, and 1.0-ft level. The average total solids concentration for the slurry was determined to be 120.5 g/L.

Zone test. The zone settling test was performed concurrently with the compression settling test on the same slurry. The depth to the interface was read approximately every 15 min. The zone test ran for approximately 12 hr. From the plot of the depth to interface (feet) versus time (hours), zone settling velocity was determined.

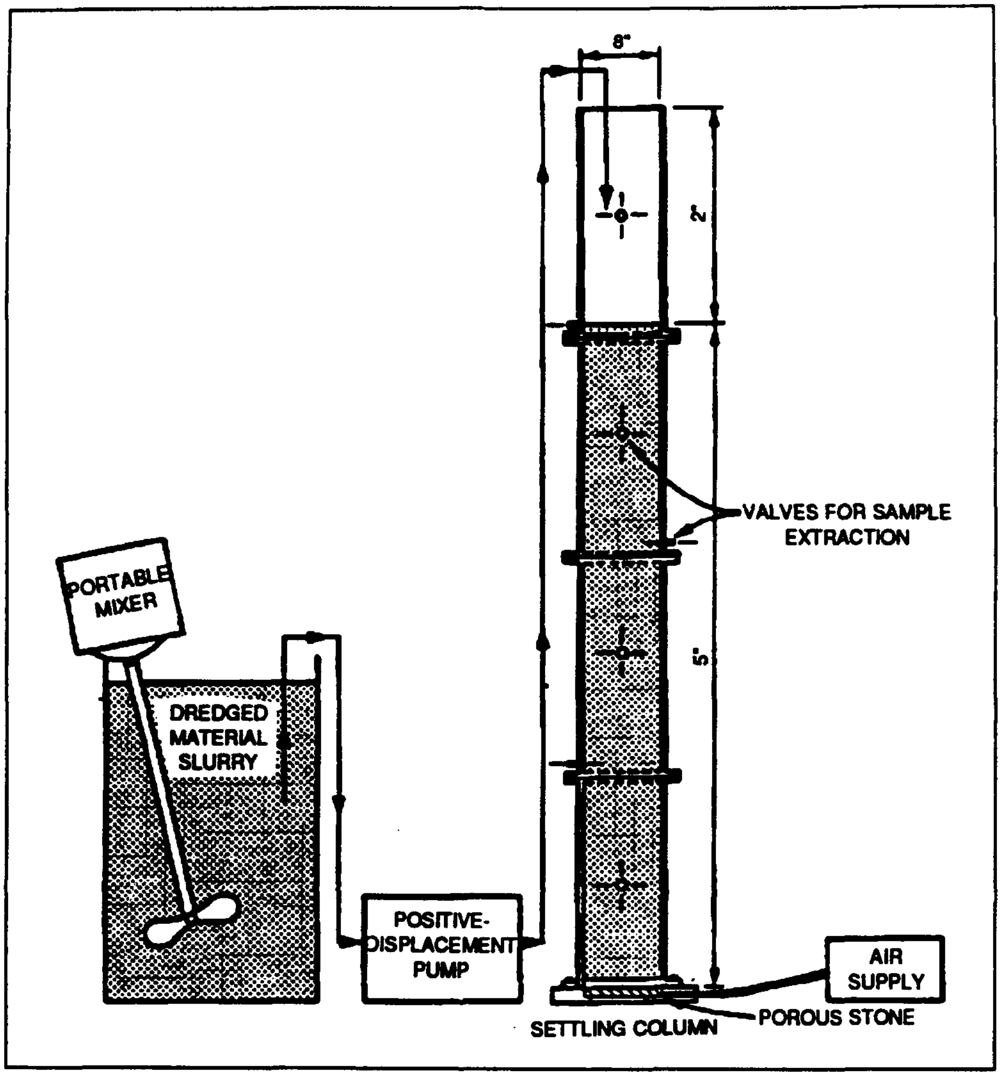

Figure 6. Schematic of the settling column (Palermo 1985)

Compression test. The depth to the interface was measured at approximately 15-min intervals for the first 8 hr and at 30-min intervals for the next 4 hr, which were the same times as those used for the zone test (as described above). Thereafter, for 15 days, depth to the interface was measured at 1- to 3-day intervals, and these data were used for the compression test.

Flocculent test. Flocculent settling tests were performed concurrently with the zone and compression settling tests on the same slurry. Therefore, the flocculent, zone, and compression settling test initial slurry concentrations were the same. Samples of the supernatant were extracted with a syringe at 6.0-, 5.5-, 5.0-, 4.5-, 4.0-, 3.5-, and 3.0-ft ports above the liquid-solid interface at different time intervals (3.0, 6.0, 8.0, 12.0, 24.0, 48.0, and 96 hr). Suspended solids concentrations were then determined on the supernatants following Standard Method 2540D (APHA-AWWA-WPCF 1989).

Modified elutriate test

The procedure for conducting a modified elutriate test, as shown in Figure 7, is described in the following paragraphs.

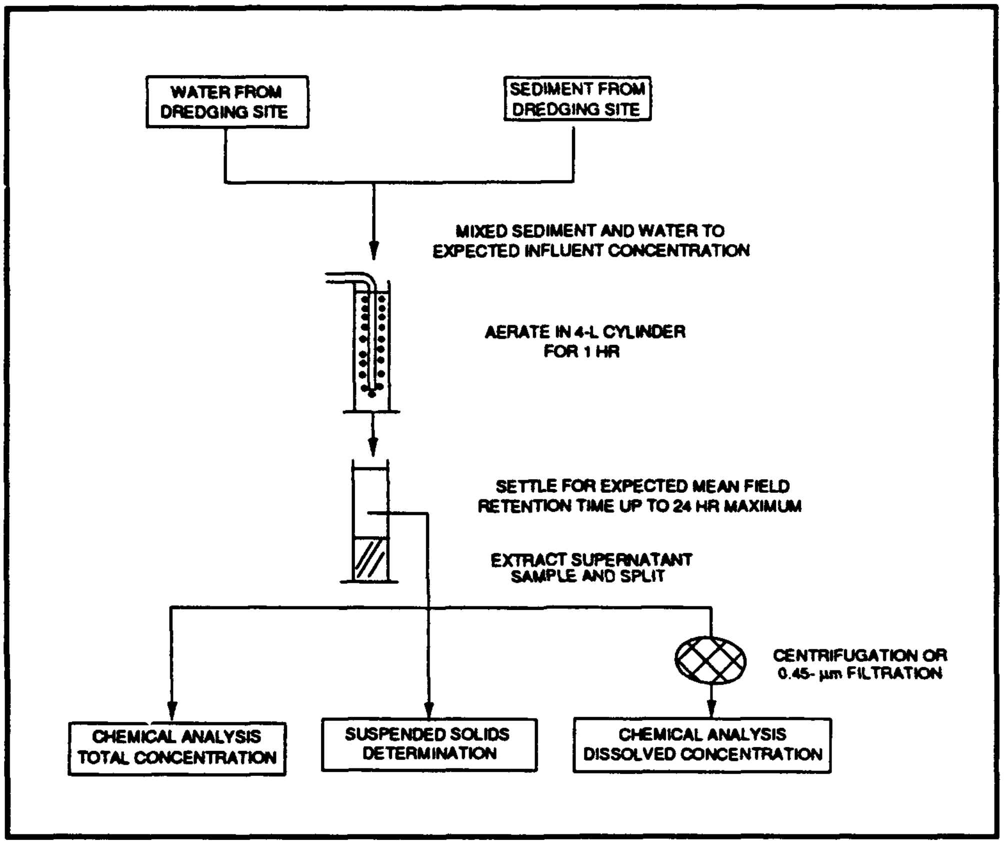

Figure 7. Modified elutriate test procedure

Apparatus and testing procedure. The modified elutriate testing apparatus consists of a laboratory mixer and several 4-L graduated cylinders. The volume required for each analysis, the number of parameters measured, and the desired analytical replication will influence the total elutriate sample volume required. The test procedure involves mixing site water and sediment to a concentration expected in the influent to a CDF. The mixture is then aerated for 1 hr to simulate the oxidizing conditions present at the disposal site. Next, the mixture is allowed to settle for a time equal to the expected or measured mean retention time of the disposal area, up to a maximum of 24 hr. The sample of the supernatant water is extracted for single analysis of dissolved and total contaminant concentrations. Detailed procedures for the modified elutriate test as conducted at WES are presented below.

Sample preparation. The sediment and dredging site water were mixed to a target slurry concentration of 150 g/L. The composite sediment concentration was 476 g/L. Each 4-L cylinder to be filled required a mixed slurry volume of 3.75 L. The volumes of sediment and dredging site water to be mixed in the cylinders were calculated using the following equations:

$$V_{sediment} = 3.75 \times (C_{slurry}/C_{sediment}) \tag{1}$$

and

$$V_{water} = 3.75 - V_{sediment} \tag{2}$$

where

$V_{sediment}$ = volume of sediment, L

3.75 = volume of slurry in a 4-L cylinder, L

C_{slurry} = desired concentration of slurry, g/L

$C_{sediment}$ = predetermined concentration of sediment, g/L

V_{water} = volume of dredging site water, L

The slurry was prepared by adding 1.18 L of sediment to 2.57 L of site water in a large container.

Mixing of the slurry. The slurry was mixed in a large container for 15 min with a laboratory mixer. The slurry was mixed to a uniform consistency.

Aeration of the slurry. Aeration was used to ensure oxidizing conditions in the supernatant water to simulate dredging operation during the mixing phase. The mixed slurry was poured into 4-L graduated cylinders. The slurry was aerated by using compressed air that passed through a deionized water trap, through a glass tubing, and bubbled through the slurry. The agitation was vigorous and continued for 1 hr.

Settling of the slurry. The tubing was then removed from the cylinder thereby allowing the aerated slurry to undergo quiescent settling for 24 hr, a suggested default value when the field mean retention time is not known.

Sample extraction. After the 24-hr settling period, samples of the supernatant water were extracted from the cylinder at a point midway between the water surface and the interface using a syringe and tubing. Care was taken not to resuspend settled material. The extracted samples were homogenized, split, and analyzed for total suspended solids concentration, dissolved contaminants, and total contaminants of selected constituents. Samples for the analysis of dissolved contaminants were filtered through a 0.45-μm filter.

Data Analysis and Results

The behavior of Conemaugh River Lake sediments at slurry concentrations equal to that expected for inflow to a CDF is governed by zone settling processes. The sediments exhibited a clear interface between settled material and clarified supernatant.

The settling test data were analyzed using the Automated Dredging and Disposal Alternatives Management System (ADDAMS) (Schroeder and Palermo 1990), which is a family of computer programs developed to assist in planning, designing, and operating dredging and dredged material disposal projects.

All chemical analyses for this study were conducted according to SW-846 standard procedures (Table 1). Metals were analyzed using one of the following instruments: Inductively Coupled Argon Plasma (ICP), Perkin Elmer 5000 (Cold Vapor), and Zeeman 5100. Cyanide analysis was performed on the Technicon Auto Analyzer. Organic analyses were performed using gas chromatograph/mass spectrometers (GC/MS). The Analytical Laboratory Group (ALG) at WES performed these analyses.

Table 1
Analytical Procedures

Parameter	Method	Reference
Metals	ICP and Furnace Methods	USEPA Method 6010 USEPA Series 7000 Methods
Semivolatile or BNAs	GC/MS	USEPA Method 8270
Pesticides and PCBs	GC	USEPA Method 8080
Cyanide	Colorimetric	USEPA Method 9012

Bulk chemistry

Homogenized samples (in triplicate) of the sediment were sent to the ALG to determine the chemical characteristics of the sediment (Tables A1 and A2). The sediment was analyzed for total metals, organic priority pollutants except volatiles, total organic carbon, and radioactivity (gross gamma, gross alpha, and gross beta). The analytical results of the sediment show elevated levels of heavy metals, such as arsenic (51 mg/kg), barium (348 mg/kg), chromium (49 mg/kg), copper (83.3 mg/kg), lead (570 mg/kg), and mercury (1.0 mg/kg). The average cyanide concentration was 625 mg/kg. The reportable organic pollutants detected were PCB-1254 (0.074 mg/kg) and 1,2,4 Trichlorobenzene (10.7 mg/kg). The total organic carbon concentration was 27,538 mg/kg. More detailed analytical results are presented in Table A1.

The results of the radionuclides activity are reported as pci/g of dry sediment (Table A2). Both gross alpha and gross beta values were typical of normal background radiation. The data reported for the gross gamma scan were within normal background levels. All radionuclides detected in the gamma scan were naturally occurring with the exception of ^{137}Cs, which is a residual remaining in the environment as a result of atmospheric testing in the 1960s.

Modified elutriate test

Since the bulk chemistry results gave a "reason to believe" that the sediment may be contaminated, the modified elutriate test was conducted on the Conemaugh River Lake sediment to evaluate the potential for contaminant releases from the CDF during dredging operations. Results for all analytes are shown in Table A3. The analytical results show total concentration and dissolved concentration of total organic carbon (TOC), cadmium, copper, aluminum, barium, iron, and manganese (Table 2). Cyanide was not detected in the modified elutriate test. Possible explanations for no detection of cyanide in the modified elutriate test are the oxidation that occurs during the elutriate test and the cyanide may be in a form that remains attached to the sediment.

Table 2
Results of Modified Elutriate Tests

Parameter	Total Concentration mg/L	Dissolved Concentration mg/L	Fraction of Total Suspended Solids mg/kg of TSS
TOC	23.4	21.8	5,630
TSS	284	–	–
Cadmium	<0.0001[a]	0.0017	0
Copper	0.001	<0.001[a]	4.0
Aluminum	0.042	<0.030[a]	148.0
Barium	0.332	0.298	120.0
Iron	0.228	0.175	187.0
Manganese	0.092	0.093	0

[a] "<" values were assigned zero.

The chemical analysis of the modified elutriate samples provided the data used to predict dissolved and total concentrations of contaminants in milligrams per liter. The total suspended solids (TSS) concentration was also determined. To predict the total concentration of each contaminant in the effluent, it was necessary to first calculate the fraction of each contaminant associated with the total suspended solids in the elutriate samples using the following equation:

$$F_{ss} = (1 \times 10^6) \times \frac{C_{total} - C_{diss}}{SS} \tag{3}$$

where

F_{ss} = fraction of contaminant in the total suspended solids, mg contaminant/kg of suspended solids

(1×10^6) = conversion factor, mg/mg to mg/kg

C_{total} = total concentration, mg contaminant/L of sample

C_{diss} = dissolved concentration, mg contaminant/L of sample

SS = total suspended solids concentration, mg solids/L of sample

The results for these calculations using Equation 3 are summarized in Table 2, which shows only the detected parameters.

Column settling tests

Compression settling tests. For the compression tests, the initial slurry concentration and height and depth to interface versus time were entered in the program (Table A4). The ADDAMS program used the initial slurry concentration of 120.5 g/L and height of 6.22 ft to determine the solids concentration at a given time. A plot was generated showing the relationship between solids concentration (g/L) and retention time (days) (Figure 8). ADDAMS also developed a regression equation for the resulting power curve relating solids concentration to time. The composite sample regression equation may be used to determine the solids concentration at any given time. The regression equation used was

$$C = 210 \times T^{0.2471} \tag{4}$$

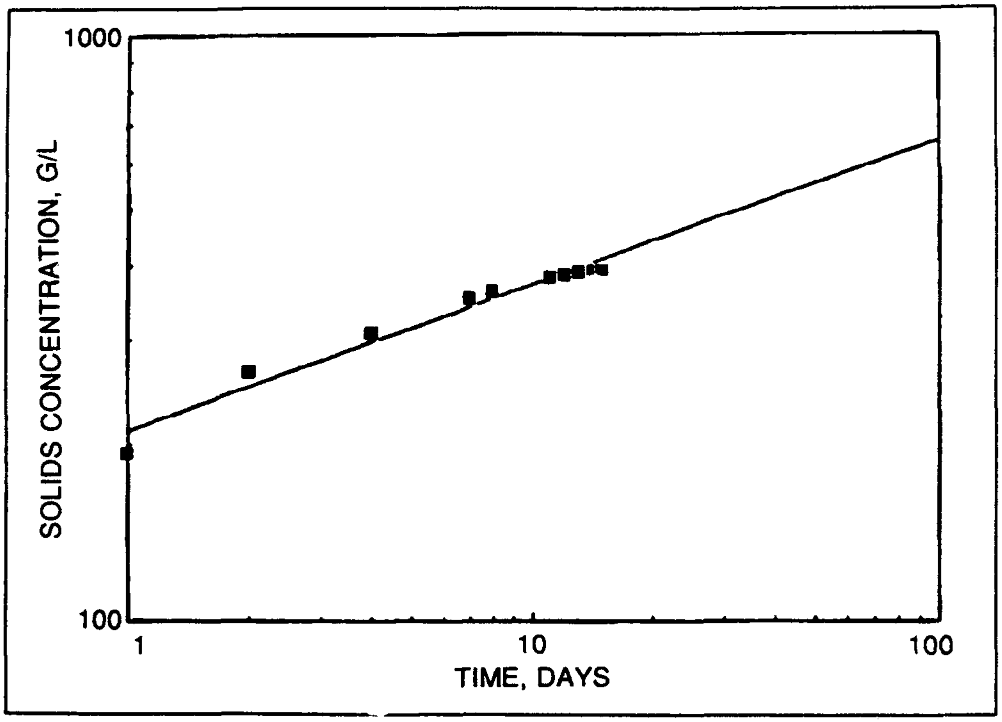

Figure 8. Compression test curve

where

C = solids concentration, g/L

T = time, days

Zone settling test. Zone settling velocity for the Conemaugh River Lake composite sample was determined to be 0.101 ft/hr for the zone test. Depths to interface and their corresponding time intervals were entered (Table A5) into a plotting routine used to determine the zone settling velocity. When the zone settling curve departs from a linear relationship, compression settling begins (Figure 9).

Flocculent settling tests. For the flocculent tests, an extension to the zone settling procedure is presented in EM 1110-2-5027. Palermo (1984) analyzed the effects of several possible assumptions regarding the magnitude of the value to be used as the initial concentration in laboratory test, and he showed that all gave essentially the same final result. Therefore, he recommended that, for simplicity, the concentration in the first sample taken at the highest sampling port be used as the initial concentration. The initial concentration and the supernatant suspended solids concentrations at different depths and time intervals (Table A6) were used by ADDAMS to generate two curves, the concentration profile curve and the supernatant suspended solids curve (Figures 10 and 11, respectively). The concentration profile curve, which plots the depth below the surface (feet) versus percent of initial concentration, shows

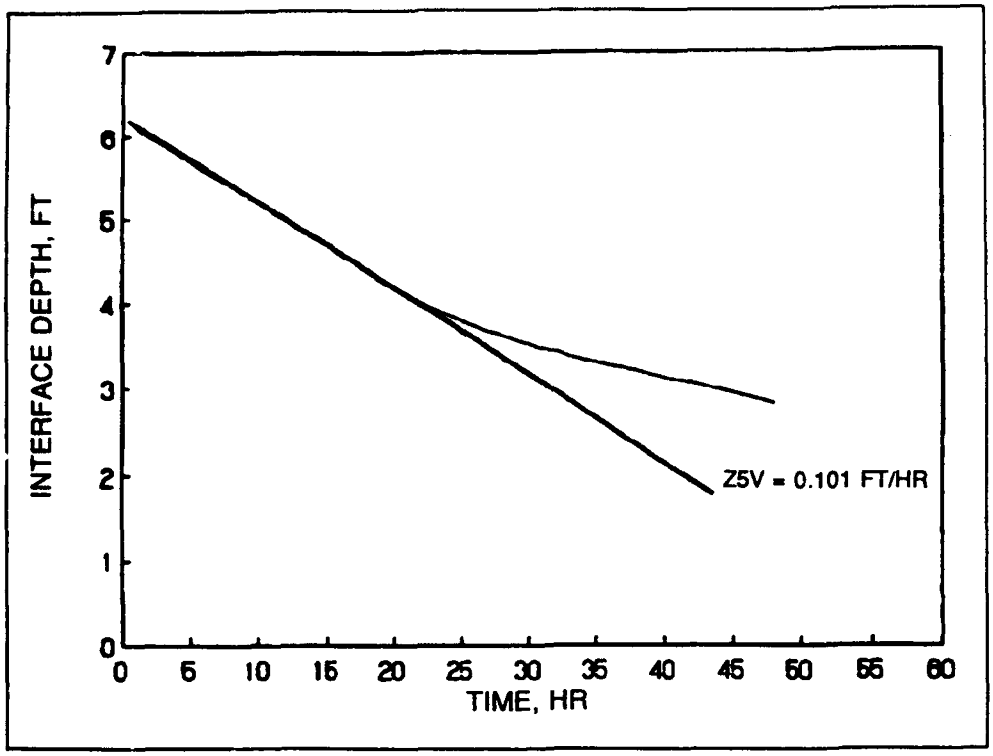

Figure 9. Interface depth versus time curve

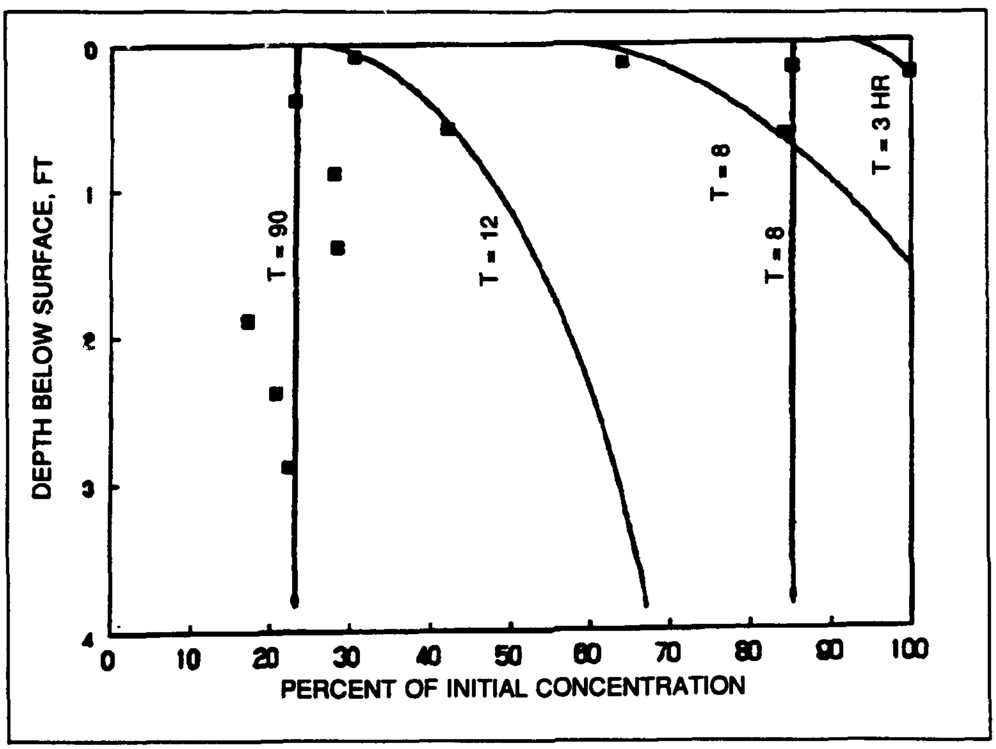

Figure 10. Solids concentration profile curve

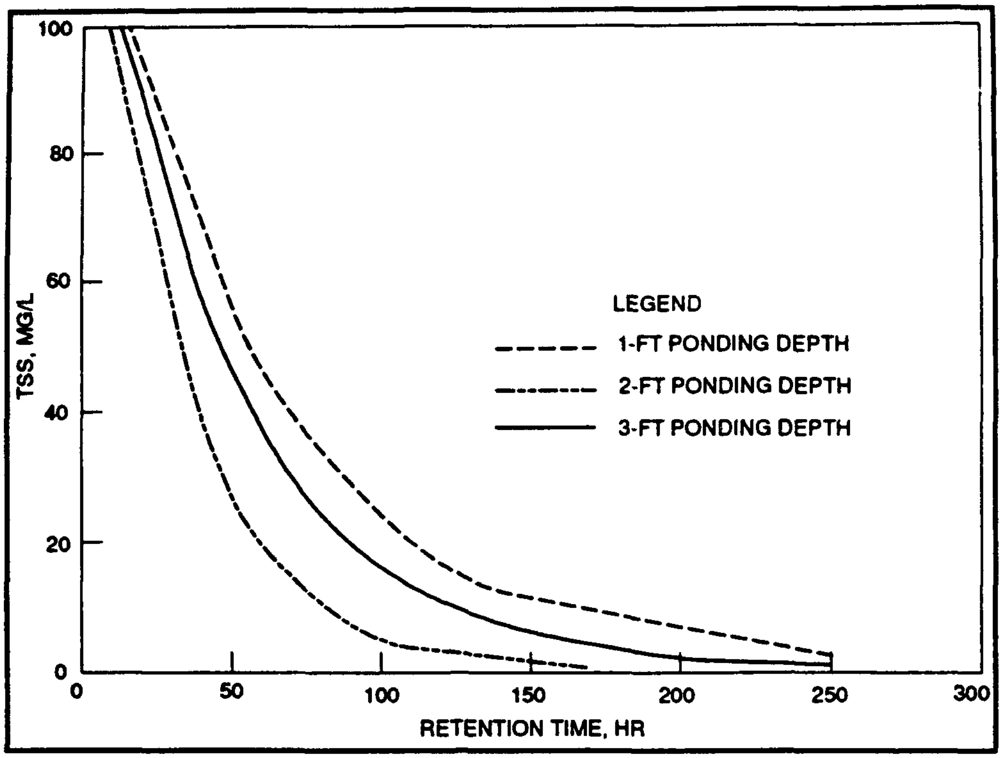

Figure 11. Supernatant suspended solids curve

that the suspended solids concentrations decrease with time and increase at deeper ponding depths (1, 2, and 3 ft) at the weir. The supernatant suspended solids curves derived from the concentration profile curves compare the effect of retention time on supernatant suspended solids at 1-, 2-, and 3-ft ponding depths. This curve shows that increasing the retention time beyond 70 hr for 2 ft of ponding depth provides little additional improvement in supernatant suspended solids concentration. Actual field suspended solids will be greater because of resuspension by wind and wave action. The resuspension factor is estimated at approximately 1.5 to 2.5 depending on ponding depth and surface area (Table 3).

Table 3
Recommended Resuspension Factors for Various Ponded Areas and Depths

	Anticipated Average Ponded Depth	
Anticipated Ponded Area	**Less than 2 ft**	**2 ft or Greater**
Less than 100 acres	2.0	1.5
Greater than 100 acres	2.5	2.0

Application of Results to Conceptual Design of a Typical CDF

Sediment characteristics

Sediment characteristics of the dredged material are important in the design of a CDF. Some sediment characteristic values at Conemaugh River Lake are listed below.

Initial water content, 170 percent
Specific gravity, 2.68
Initial void ratio, 4.55
Percent sand (avg), 9.9

The predominant Unified Soil Classification is organic clay (OL) (Figure 12).

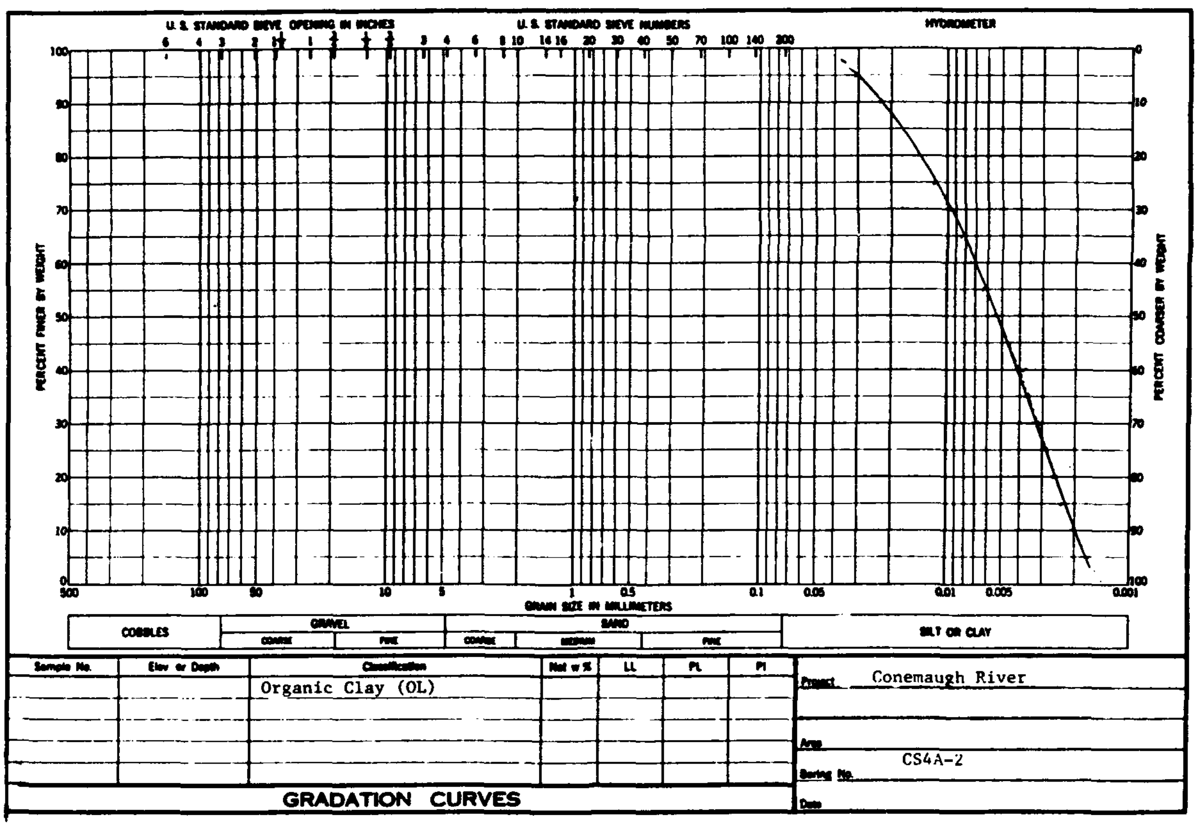

Figure 12. Grain size analysis curve, Conemaugh River Lake sediment

Typical project conditions

The conditions surrounding the Conemaugh River Lake may hinder the removal operation of dredged material. The area adjacent to the dam will be hindered by the presence of logs and small debris. Also, access to the work site is severely limited and will restrict the size of equipment that can be mobilized for construction. It is understood that the 2.5-ton load limit on the former railroad bridge providing access to the left bank is being reviewed as a result of repairs made in the late 1980s in preparation for construction of the non-Federal hydropower development project.

The only area available for serious consideration as a potential disposal facility for dredged material near the damsite is across the inside of the oxbow just upstream of the dam on the left descending bank (Figure 13). Due to the steep banks, the potential site would have limited storage capacity. Pittsburgh District personnel have estimated that developing a usable volume of 150,000 yd^3 will require construction of enclosure dikes 20 ft high. A 10-ft-high dike will provide a usable volume of 40,000 yd^3. A review of sedimentation surveys has indicated that during the period 1966 to 1982, 320,000 yd^3 of material have been deposited.

Figure 13. Aerial view of oxbow, potential CDF site

Preliminary design of a CDF also requires knowledge of additional project conditions. The dredge production rate, dredge flow rate, site capacity, dike height, sediment storage depth, ponding depth, and freeboard depth are needed (Wade 1988). For the purpose of illustrating how to use the information developed in this study, the following project conditions are assumed:

(a) volume to dredge is 350,000 yd^3; (b) since access to the site is limited, an 8-in. dredge is used and expected to dredge at an effective production rate of 175 yd^3/hr; (c) dredged material slurry flow rate is 700 cu yd^3/hr (5.25 cfs) for a slurry concentration of 120 g/L; and (d) the dike, storage, ponding, and free-board depths are 10, 6, 2, and 2 ft, respectively.

Design of CDF

Because dredged material has the tendency to increase in volume, the actual amount of dredged material from Conemaugh River Lake requiring storage may be larger than 350,000 yd^3. The total volume required for initial storage in a containment area includes volume for storage of dredged material, volume for sedimentation (ponding depth), and freeboard volume (volume above water surface). The volume required for storage of the coarse-grained material (>No. 200 sieve) is determined separately because this material behaves independently of the fine-grained material (<No. 200 sieve). Design computations are as follows:

a. Representative samples of channel sediments tested in the laboratory indicate that 9.9 percent of the sediment is coarse-grained material (>No. 200 sieve). Therefore

$$V_{sd} = 350{,}000(0.099) = 34{,}650 \text{ yd}^3$$

$$V_i = 350{,}000 - 34{,}650 = 315{,}350 \text{ yd}^3$$

b. Estimate the time of dredging:

$$\frac{350{,}000 \text{ yd}^3}{175 \text{ yd}^3/\text{hr}} = 2{,}000 \text{ hr}$$

Since, the estimated time of dredging is 2,000 hr, two 8-in. dredges will be used where operating time per day per dredge is 18 hr. Thus,

$$\frac{2{,}000 \text{ hr}}{36 \text{ hr/day}} = 56 \text{ days}$$

(1) Average time for dredged material consolidation:

$$\frac{56 \text{ days}}{2} = 28 \text{ days}$$

(2) Design solids concentration of settled solids at 28 days:

$C_d = 210 \times T^{0.2471}$ (from Figure 8)

$C_d = 478$ g/L

c. Estimate the volume required for dredged material:

(1) The average void ratio of the fine-grained material is calculated as follows:

$$e_o = \frac{G_s \times 1{,}000}{C_d} - 1 \tag{5}$$

$$e_o = 4.60$$

(2) The volume of the fine-grained material after disposal is calculated as follows:

$$V_f = V_i \left[\frac{e_o - e_i}{1 + e_i} + 1 \right] \tag{6}$$

where

V_f = volume of fine-grained material after disposal in CDF, ft^3

V_i = volume of fine-grained channel sediments, ft^3

e_i = initial void ratio in sediment

$$V_f = 317{,}619\ yd^3$$

(3) The volume required for initial storage is calculated as follows:

$$V = V_f + V_{sd} \tag{7}$$

where

V = total volume of the dredged material in the CDF, yd^3

V_{sd} = volume of sand, yd^3

$$V = 317{,}619 + 34{,}650$$

$$= 352{,}269\ \mathrm{yd}^3$$

d. Determine the maximum thickness of dredged material at end of disposal operation.

(1) The dike height is limited to 10 ft. The allowable dredged material height is calculated as follows:

$$H_{dm(max)} = H_{dk(max)} - H_{pd} - H_{fb} \tag{8}$$

where

$H_{dk(max)}$ = maximum allowable dike height, ft

H_{pd} = ponding depth, ft

H_{fb} = freeboard (minimum of 2 ft can be assumed), ft

$$H_{dm(max)} = 10 - 2 - 2$$

$$= 6 \text{ ft}$$

(2) The minimum possible surface area is calculated as follows:

$$A_{ds} = \frac{V}{H_{dm(max)}} \tag{9}$$

$$A_{ds} = \frac{352{,}269 \text{ yd}^3 \times 27 \text{ ft}^3/\text{yd}^3}{6 \text{ ft}}$$

A_{ds} = 1,585,209 ft^2

A_{ds} = 37 acres

e. Determine minimum area required for zone sedimentation.

(1) V_s = 0.101 ft/hr (from Figure 9).

(2) The area requirement is calculated as follows:

$$A_z = \frac{Q_i\,(3{,}600)}{V_s} \tag{10}$$

$$Q_i = 5.25 \text{ cfs} \times 2 = 10.5 \text{ cfs} \tag{11}$$

$$A_z = \frac{10.5 \times 3{,}600}{0.101}$$

$$= 374{,}250 \text{ ft}^2$$

$$A_z = 8.6 \text{ acres}$$

where

A_z = containment surface area requirement for zone settling, ft^2

Q_i = influent flow rate, cfs

3,600 = conversion factor, hours to seconds

V_s = zone settling velocity at influent solids concentration (C_i), ft/hr

(3) Increase the area by a factor of 1.87 [hydraulic efficiency correction factor (HECF)] to account for hydraulic inefficiencies (assuming the CDF can be constructed with a length-to-width ratio of approximately 3):

$$T_d / T = 0.9 \; [1 - \exp(-0.3L/W)] \tag{12}$$

$$= 0.53 \text{ (assuming length-to-width ratio is 3)}$$

and

$$T_d / T = 1/\text{HECF}$$

therefore

$$\text{HECF} = 1.87$$

$$A_{dz} = 1.87 \times (8.6 \text{ acres})$$

$$A_{dz} = 16.0 \text{ acres}$$

where

A_{dz} = design surface area for effective zone settling, acres

HECF = hydraulic efficiency correction factor

T_d = mean residence time, hr

T = theoretical residence time, hr

f. Determine minimum area required for ponding.

(1) The effluent suspended solids concentration was predicted as follows:

$$\text{Total settling volume} = 352{,}269 \text{ yd}^3 \times 2 \text{ ft}/6 \text{ ft}$$
$$= 117{,}000 \text{ yd}^3$$

$$T = 117{,}000 \text{ yd}^3 \div 25{,}200 \text{ yd}^3/\text{day}$$

$$= 4.66 \text{ days (112 hr)}$$

The hydraulic efficiency factor is applied due to containment area inefficiencies (Shields, Schroeder, and Thackston 1987).

$$T_d = 112 \text{ hr} \div 1.87$$

$$= 60 \text{ hr}$$

The supernatant suspended solids curve (Figure 11), a retention time of 60 hr, and a 2-ft ponding depth yield a suspended solids concentration of 40 mg/L in the column. A resuspension factor of 1.5 is recommended for a ponding depth of 2 ft or greater and a surface area less than 100 acres. The effluent suspended solids concentration estimated for the field conditions is 60 mg/L.

$$A_{df} = \frac{T\ Q_i}{H_{pd}\ (12.1)} \qquad (13)$$

$$= \frac{112 \times (10.5)}{2 \times (12.1)}$$

$$A_{df} = 49 \text{ acres}$$

The CDF site should therefore encompass approximately 49 acres of ponded surface area if the dredge selected for the project has an effective flow rate not greater than 5.3 cfs. In this case, the surface area of 49 acres required to meet effluent suspended solids concentration of 60 mg/L is greater than the minimum surface area of 16 acres required for effective zone settling. The area required for storage is 37 acres. The design surface area A_d is therefore 49 acres as required for ponding. This corresponds to the following values as previously calculated:

$H_{dm} = 6$ ft
$H_{pd} = 2$ ft
$H_{fd} = 2$ ft
$A_d = 49$ acres

Predicted effluent suspended solids concentrations

After the dredged material is placed in a CDF, solids that have not settled by gravity will remain suspended in the water column. The solids that are suspended will flow over the weir structure. The concentration of the suspended solids in the effluent is needed to determine the effectiveness of the CDF and if any water quality standards will be violated.

Prediction of the total contaminant concentrations in the effluent were made using the results of the modified elutriate test and column settling test. The total contaminant concentrations in the effluent were predicted by adding the predicted dissolved concentrations and the predicted particle-associated concentrations. The dissolved concentrations were determined directly by the modified elutriate test. The particle-associated concentrations were calculated using the contaminant fractions (Table 2) of the total suspended solids determined by the modified elutriate test and the predicted effluent suspended solids concentration determined by the column settling test. Both test results were used to predict total contaminant concentration in milligrams per liter in the effluent by using the following equation (Thackston and Palermo 1990):

$$C_{total} = C_{diss} + \frac{F_{ss} \times SS_{eff}}{(1 \times 10^6)} \qquad (14)$$

C_{total} = estimated total concentration in effluent, mg contaminant/L of water

C_{diss} = dissolved concentration determined by modified elutriate test, mg contaminant/L of sample

F_{ss} = fraction of contaminant in the total suspended solids calculated from modified elutriate test results, mg contaminant/kg of suspended solids

SS_{eff} = predicted suspended solids concentration of effluent estimated from evaluation of sedimentation performance in laboratory column settling test, adjusted for field conditions by factors from Table 3 (Palermo and Thackston 1988), mg suspended solids/L of water

(1×10^6) = conversion factor, mg/mg to mg/kg

Table 4 shows the predicted total concentration of possible contaminants in the effluent. The acceptability of the proposed CDF operation can be evaluated by comparing the predicted total contaminant concentrations with applicable water quality standards. Since no water quality standards were specified, predicted concentrations of contaminants in the effluent were compared with Federal Water Quality Criteria (WQC) for surface water. All predicted effluent concentrations were below the WQC except cadmium, which was slightly higher than the WQC; therefore, no mixing zone evaluation was required.

Table 4
Comparison of Predicted Effluent Quality and Drinking Water Standards

Parameter	Predicted Total Concentration in Effluent, mg/L	Federal Water Quality Criteria[1]	
		Fresh Acute Criteria mg/L	Fresh Chronic Criteria, mg/L
TOC	22.1	--	--
TSS	--	--	--
Cadmium	0.0017	0.0039	0.0011
Copper	0.001	0.018	0.012
Aluminum	0.032	--	--
Barium	0.304	--	--
Iron	0.184	--	1
Manganese	0.093	--	--

[1] U.S. Environmental Protection Agency, Office of Water Regulations and Standards, September 1986.

3 Conemaugh River Lake Numerical Model

Introduction

Purpose and approach

As stated previously, since 1952 significant sedimentation has occurred just upstream from the dam. The sedimentation is on the order of 20 to 25 ft above the low level outlets and 30 to 35 ft above the original channel bed elevation (Figure 3). This sedimentation, while not a large percentage of total storage (4.14 percent in 1982), is critically located in the channel above the dam and is beginning to interfere with the operation of the outlet works.

The installation of the hydropower project in the late 1980s has raised concerns of future sedimentation problems. Much of the water that was previously released through the dam is now released through the powerhouse turbines. The turbine inlet is approximately 1.8 miles upstream from the dam, and the release of clear water is expected to worsen sedimentation problems in the channel downstream of the turbine intake.

Plan description

The alternatives tested in this study consisted of (a) the existing condition with no change in operation (no action); (b) increased minimum flow through the dam during low flow periods; and (c) removal of sediment deposition from the pool by dredging with no change in operating procedure. Additionally, the Pittsburgh District proposed a fourth alternative for testing after preliminary review of model test results from the first three alternatives. This fourth alternative, described in detail later in this report, was similar to the third alternative except that the reservoir operating procedure was modified.

Sediment Analysis

Five bore samples (as shown in Figure 2) were analyzed to obtain bulk density, particle size, settling, consolidation, and shear stress characteristics. The five borings were provided by the Pittsburgh District through the WES EL to the HL for examination and density determination. The remainder of the analyses were performed on a single sample (CS4A).

Sample conditions and density

Samples were sealed in 5-ft PVC tubes. Sample CS4A-2 was found to be a full sample with very little water standing on it, and was homogenized with a drill motor for 15 min. Other samples had substantial standing water, and sediment material was subsampled to determine density. Results were as follows:

Sample No.	Length of Sediment in Tube, in.	Bulk Density, g/cu cm
CS4A-2	60	1.354
CS4A-1	48	1.173
CS4-1	3	1.257
CS2-1	10	1.187
CS1-1	18	1.303

All samples contained organic matter (leaves, wood fibers, twigs, etc.) which was removed by passing all samples through a 60-mesh sieve. Samples had a heavy organic smell resembling diesel fuel.

Methods

Density was measured with a Parr DMA digital density meter. Bulk wet density (BWD, g/cm^3) can be converted to solids content (C_s, g/cm^3) by use of the following equation:

$$C_s = \frac{p_s(BWD - p_1)}{p_s - p_1}$$

where p_s and p_l are the particle and liquid densities (assumed to be 2.65 and 0.997 g/cm^3, respectively).

Particle size was determined using a Particle Data ELZONE 80XY particle size analyzer that operates on a principle similar to a Coulter Counter. The machine measures spherical-equivalent diameter by the displacement of current flowing through an electrolyte in a small orifice. Samples were passed through a No. 200 sieve. Samples were diluted about 1:10,000 and suspended in 1 and 2 percent sodium chloride (NaCl) and drawn through the orifice. All samples were first partially analyzed, and then sample CS4A-2 was analyzed completely. The complete analysis included blending 240- and 95-µm-orifice results, and extrapolating below 1.4 µm.

Settling tests were performed in a 1.85-m-long by 10-cm-diam Plexiglas tube. Samples were passed through a No. 200 sieve. Sediments were mixed with tap water and allowed to equilibrate for a minimum of 12 hr. Tests were performed using the pipette method. Samples were drawn over a 4-hr period.

Hindered-settling consolidation (sometimes called zone, flocculent, batch, or phase II settling) was performed in 2-L graduated cylinders with a diameter of about 7 cm. Five tests were run for 20 to 160 g/L initial solids content (C_s). Interfaces between the sediment and clear supernatant were monitored over a 48-hr period. The initial descent rate of the interface was used to estimate hindered settling rates for the initial C_s, and the C_s at short times (about 30 min) were used as an indication of newly deposited bed density.

Shear stress tests were performed with a Rheology International Series 2 viscometer to gauge the shear strength of the sediment. A concentric cylinder geometer was used. Five test series were performed at 1.354, 1.30, 1.25, 1.20, and 1.15 g/cm^3. Shear rates were stepped up during the tests. Yield stresses were determined by extrapolation back to zero shear.

Results

During the initial sample screening for particle size, statistics based on particle number were obtained. These data are useful only for intercomparison. Results using a 95-µm orifice were as follows:

Sample	By Count in 1% NaCl		
	Mean, µm	**Median, µm**	**Mode, µm**
CS4A-2	0.09	1.94	1.43
CS4A-1	2.20	2.04	1.43
CS4-1	2.37	2.25	1.43
CS2-1	2.25	2.14	1.43
CS1-1	2.09	1.99	1.43

Sample	By Count in 2% NaCl		
	Mean, µm	Median, µm	Mode, µm
CS4A-2	2.49	2.25	1.43
CS4A-1	2.49	2.31	1.43
CS4-1	2.37	2.20	1.43
CS2-1	2.55	2.43	1.43
CS1-1	2.37	2.25	1.43

Again, the above-listed data are based on counts, which of course skews the statistics toward finer particles, and should only be used for comparison. The results indicate the particle sizes of the borings were similar, with CS4-1 and CS2-1 slightly coarser than the rest.

Results for the analysis of sample CS4A-2 are shown in the cumulative and differential gradation curves, Figures 12 and 14. The median size based on volume or mass was about 5.2 µm, with a geometric mean of 6.23 µm and a geometric standard deviation of 2.67. The following tabulation also describes the distribution.

%>	D, µm
96	1.59
84	2.34
50	5.11
16	16.58
5	30.84

The secondary peaks on the coarse end of the differential plot repeated for both orifices and are probably real. A mixture of sediments or organic particles is therefore suggested.

Six settling tests were performed. Raw data as percent removed or settled versus natural log of time in minutes are shown in Figure 15. At initial concentrations (C_o) of 41, 102, and 259 mg/L, settling was slow, and 50-percent removal was not quite reached in 4 hr. Results were extrapolated to estimate median settling velocities and were found to be similar in this concentration range. Settling rates at higher initial concentrations were greater and increased with concentration. Figure 16 shows a plot of median settling velocities versus concentration and a suggested relationship. Settling velocity (W_s) was constant at concentrations below 260 mg/L at 0.89 mm/sec. At concentrations between 260 and 1,800 mg/L, W_s(mm/sec) = 5.5e-4 C(mg/L)$^{4/3}$. At concentrations of 1,800 to 12,000 mg/L, W_s was probably about constant at 1.25 mm/sec. Hindered settling began above 12,000 mg/L.

Raw data from the consolidation tests are shown in Figure 17. Breaks in the lower two concentrations suggest that initial concentrations of deposited material may be on the order of 80 g/L. The behavior of the material at the

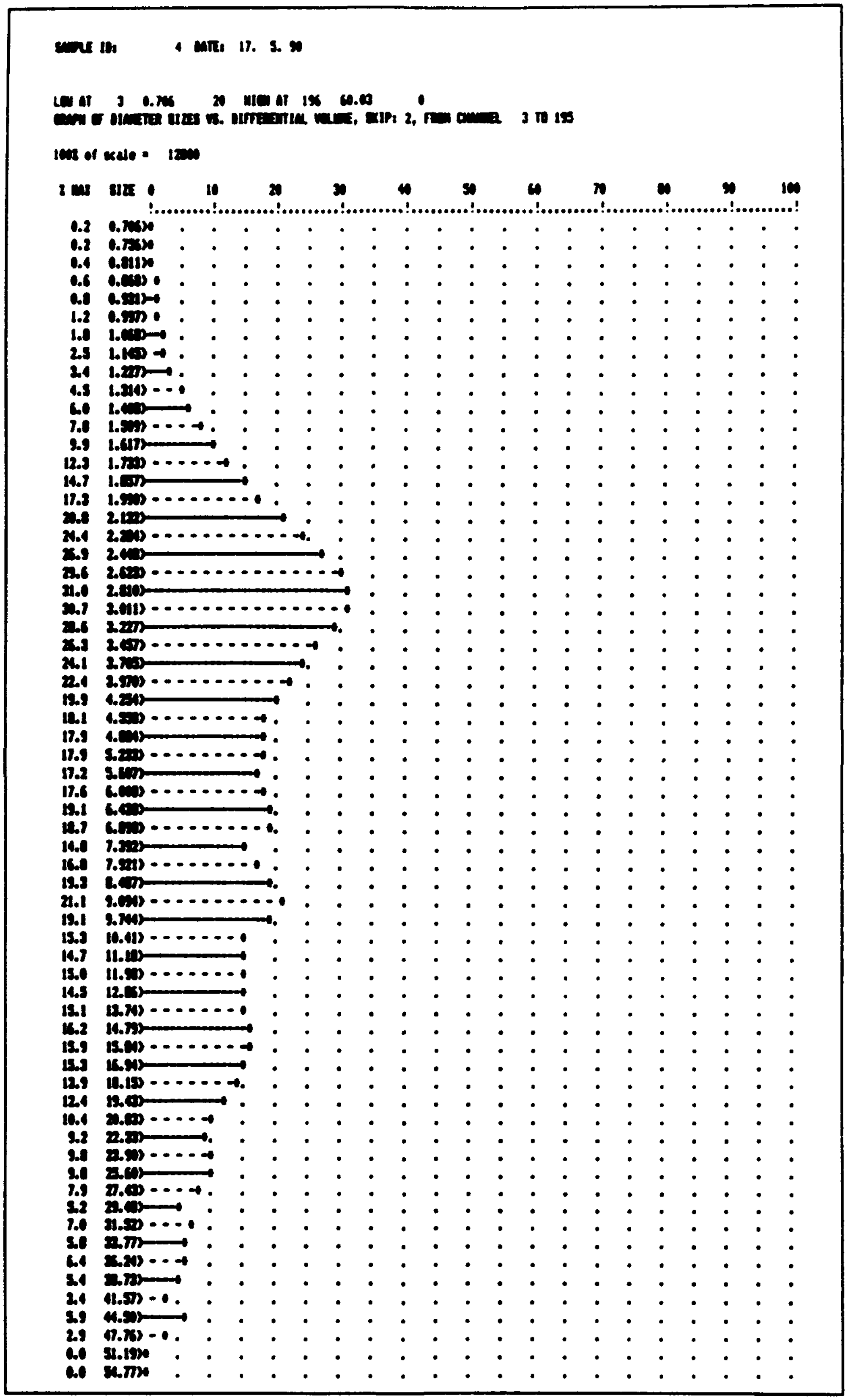

Figure 14. Differential gradation curve for sample CS4A-2

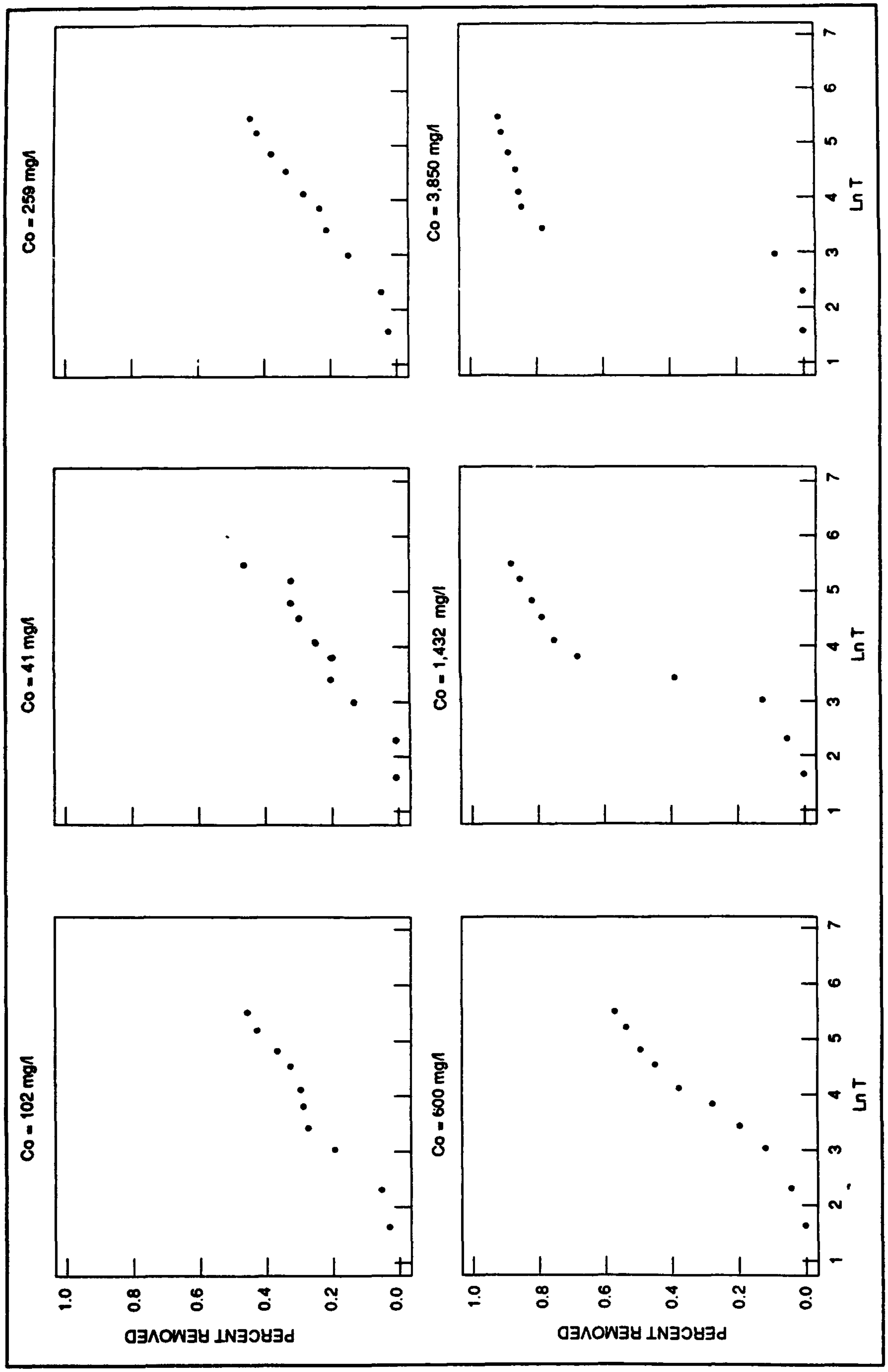

Figure 15. Percent removal versus natural logarithm of time curves

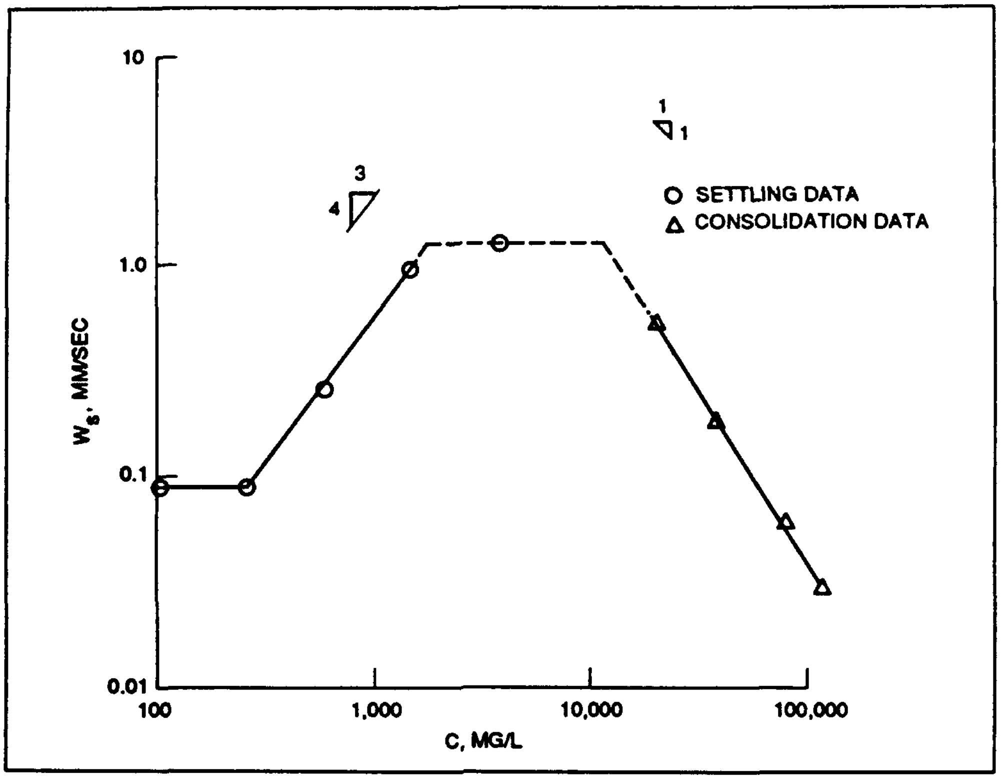

Figure 16. Median settling velocity versus concentration curve

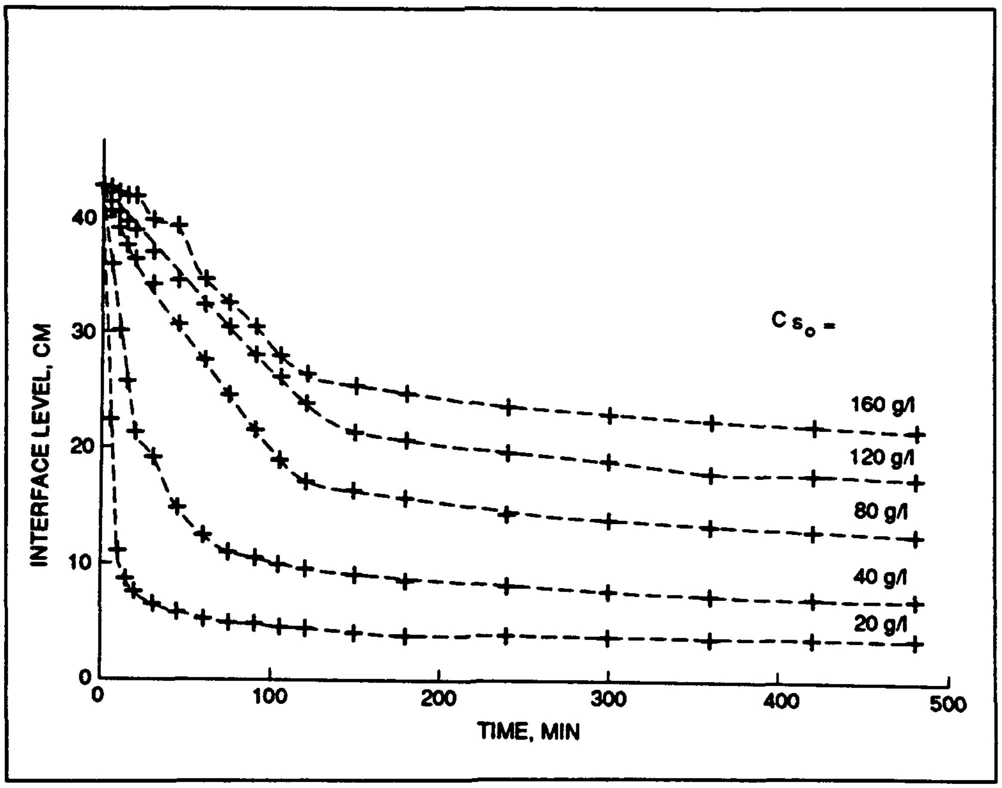

Figure 17. Consolidation curves

beginning of the highest concentration test suggests that the highest order floc density may have been reached at about 150 g/L.

Shear stress tests were performed at the in situ density of 1.354 g/cm^3 and at densities of 1.3, 1.25, 1.20, and 1.15 g/cm^3 (corresponding to 486, 405, 325, and 245 g/L). The in situ density had a consistency too great for the viscometer. Results for the low shear portion of the remaining tests are shown in Figure 18, along with linear regression lines. Results for Cs = 486 g/L were near the limit of the instrument and were somewhat erratic. Shear stress was plotted as dynes/sq cm, and 10 dynes/sq cm = 1 N/sq m = 1 pascal (Pa).

The extrapolation of the data to 0.0 shear indicated a yield stress for the material. A further extrapolation to low concentrations can indicate the critical shear stress for deposition. The assumption is that at densities equivalent to the highest order floc, critical shear stresses for erosion and deposition become equal. Based on the estimates of highest order floc density, the critical shear stress for deposition is in the density range of 0.04 to 0.06 Pa.

The sediment was found to have high shear strength. Of Krone's 1963 data, it was most similar to San Francisco sediments (Krone 1963). Figure 19 shows the present results, Krone's results, and previous flume test results for San Francisco sediments. The flume and viscometer data for San Francisco sediments do not match very well, possibly because of the time scale of the measurements. The flume tests lasted 6 hr, and it was found that erosion was

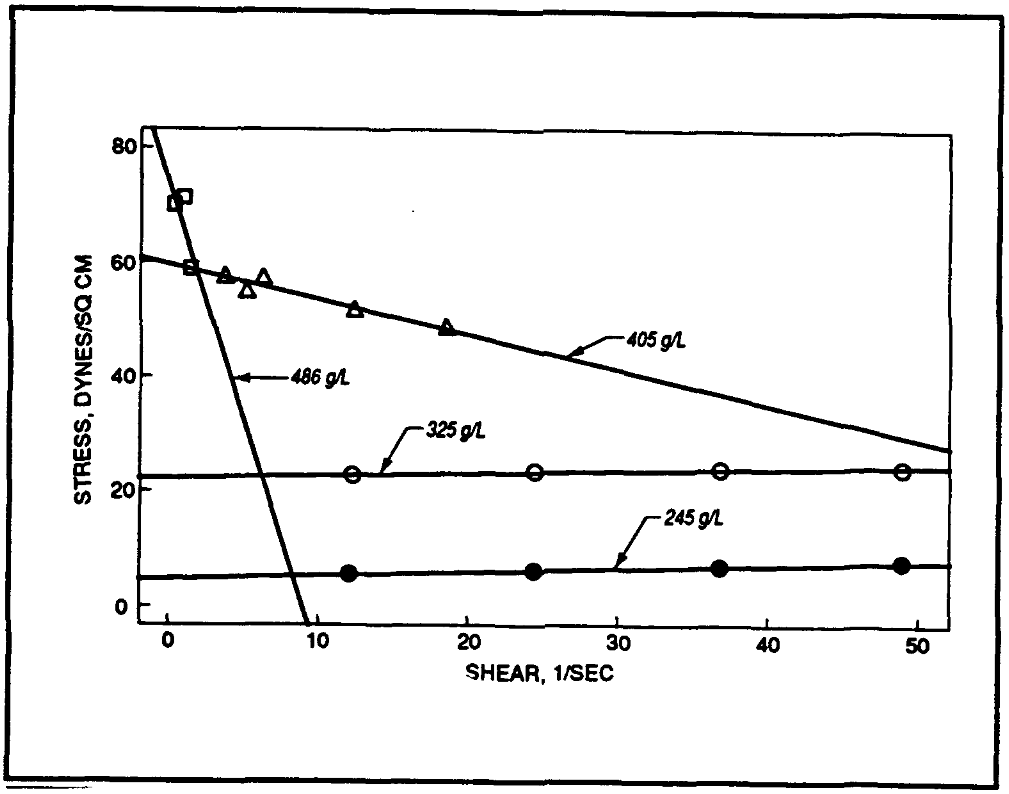

Figure 18. Shear stress curves

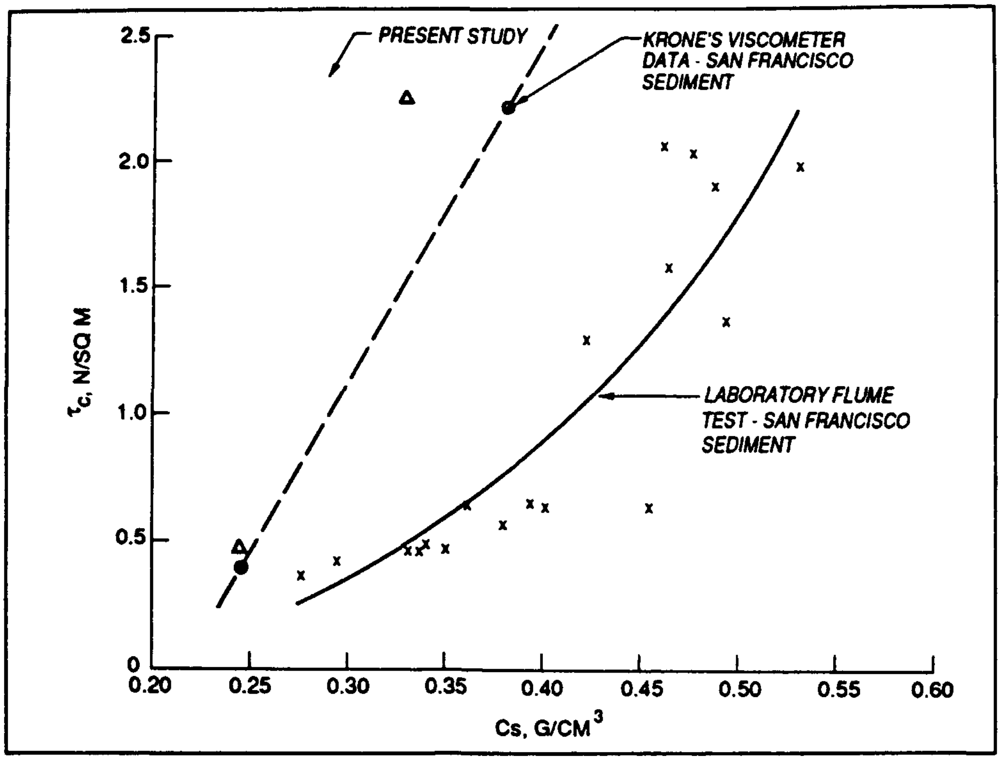

Figure 19. Comparison of τ_c versus γ_i for both sediments

sporadic and increased over time. Viscometer tests are short duration, and the material does not have the opportunity to fail in the same way (creep, cracking, and pitting). Thus, the authors would suggest using a critical shear stress (τ_c) function similar to that developed for San Francisco sediments:

$$\tau_c(\text{Pa}) = 18\ \text{Cs}(\text{g/cu cm})^{3.3}$$

The Models

Two main computer codes were utilized in the modeling of the Conemaugh River Lake and its main tributaries. The computer codes consisted of UNET, a one-dimensional unsteady hydraulic model, and TABS-1, a one-dimensional steady sedimentation model.

The UNET one-dimensional unsteady flow model was used to develop flow rates at each section used in this study. The UNET computer program developed by Dr. Robert Barkau, formerly of the U.S. Army Engineer Hydrologic Engineering Center (USAEHEC), is licensed for use within the Corps of Engineers by a Corps-wide license (USAEHEC 1991a). The program handles unsteady flows from multiple river sources through channels and/or reservoirs. The processor uses an implicit finite difference solution technique and produces a one-dimensional model that simulates an unsteady hydrograph passing through a channel network and/or reservoir. The UNET model was used in order to capture the system dynamics, which could not be modeled using either a Standard HEC-6 or TABS-1 model.

The TABS-1 one-dimensional sedimentation program was used to develop the numerical model for this study. Development of this computer program was initiated by Mr. William Thomas at the U.S. Army Engineer District, Little Rock, in 1967. Further development at USAEHEC by Mr. Thomas produced the HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs (USAEHEC 1991b). Additional modification and enhancement to the basic program by Mr. Thomas at the WES led to the TABS-1 program currently in use. TABS-1 is considered to be experimental in that it is not documented to the point that it can be made available for general use, but can be made available by special request. The program produces a one-dimensional model that simulates a series of steady-state discharge events and their effect on the sediment transport capacity at cross sections and the resulting degradation or aggradation.

Model Adjustment

UNET adjustment

The UNET unsteady flow model was adjusted using USGS- and Corps-supplied data. Average daily flow data from the USGS gauging stations at Graceton, Josephine, and Seward were used as the basis for inflow and daily reservoir stage data were supplied by the Pittsburgh District. The records from the Graceton and Josephine gauges were combined and used as inflow for Blacklick Creek (Figure 2). Data from the Seward gauge was used as inflow for the Conemaugh River. Inflow data from the Graceton and Josephine gauges were adjusted by a factor of 1.6 to account for ungauged contributions to the reservoir inflow. The UNET model was then run in 5- to 6-year increments to simulate the entire 30-yr period of record.

Initial attempts at model adjustment used the reservoir releases as the downstream boundary condition for the UNET model. The calculated water surface elevation in the dam tended to drift away from the observed values due to volume differences between the adjusted reservoir inflow and observed outflow. To eliminate the tendency to drift away from the observed data, the observed water surface elevation was used as the downstream boundary condition. The result was that while the calculated reservoir release may be different from the observed data for a day or two, due to storms on ungauged areas, etc., the model very rapidly recovered to give good agreement between calculated and observed values.

The flashy nature of the Conemaugh River and Blacklick Creek presented problems for the UNET model. The flow in both the Conemaugh River and in Blacklick Creek would vary from 500 cfs on one day to as much as 50,000 cfs for the next day during large storm events. During these large storm events, the UNET model would become unstable and/or predict negative flows at the dam. A "negative flow" signifies that water is flowing in the upstream direction. In order to keep the model stable, some of the large flood events were smoothed such that flow the day prior to the event was increased by about 1,000 cfs and flows at the peak of the event reduced by a like amount. This approach conserved mass and allowed the model to run for the entire 30 years of the simulation.

In comparing the flow rates and volumes calculated by the UNET model with the observed values, it was noted that the net volume of water released from the reservoir as predicted by the UNET model was very close to the observed value. However, it appeared that the pool was rising in the model before the water from the storms reached the lower portion of the reservoir. This resulted in negative flows at the dam, and an amount of water equal to the negative flow was then discharged from the reservoir in addition to the amount actually released due to the storm events.

The net release volume for the major storm events as calculated by the model (when negative and positive flows were added together) was very close to that observed at the project site. This indicated a phasing problem in the model. In order to reduce the number of negative flows predicted by the model, the pool elevation data were shifted from -1 to +2 days. In this process, it appeared that shifts other than a full day caused no difference in results produced by the UNET model. The optimum shift was a 1-day lag of pool elevation. This 1-day lag produced the least number of negative flow values.

After the minimum number of negative values was obtained, the data were manually smoothed to remove all negative flow values since the modified TABS-1 model could not accept negative flows for its calculations. Negative flows occurred near the dam and at the confluence of the Conemaugh River and Blacklick Creek. In order to remove the negative flows from the UNET output, the pool elevation was adjusted such that the rise in pool elevation matched the inflow of water during an event. This normally resulted in adjusting the pool elevations for 1 or 2 days. Of the 30 years modeled during the simulation, approximately 80 to 100 days were adjusted.

The adjustment of flows to eliminate the negative flows in the UNET solution should not have adversely affected the final solution, as the flows during these adjustment periods were in the lower flow regimes that immediately preceded or followed a significant event. During these low flow periods, the velocities in the areas involved were very low, and the amount of sediment carried during the 1 or 2 days involved in each adjustment was insignificant when compared with the amount carried during peak flows. The adjustments should move the solution closer to the actual trend rather than introducing additional errors. It is suspected that the negative flows are due, at least in part, to UNET's inability to account for data shifts of less than a full day.

Minor negative flows at the confluence of the two streams and upstream from the dam were removed during conversion of data to the format required by the adapted TABS-1 model. The minor negative flows (those less than 100 cfs) were corrected by taking the absolute value of the flow. Values between 100 and 1,000 cfs were set to a positive flow of 100 cfs, while those greater than 1,000 cfs were corrected by manipulating pool elevations. It should be noted that the process of manipulating pool elevations was extremely time-consuming, sometimes requiring a full-day effort to correct 4 to 6 years of data depending on the number of negative values.

The calculated UNET flow rates at each cross section in the model were then formatted as input data for the modified TABS-1 model and were used for all base and plan simulations.

TABS-1 model adjustment

The TABS-1 sediment model was adjusted using observed sediment discharge rating curves obtained from the Pittsburgh District (see Figures 20 and 21). The observed inflowing sediment concentrations were not analyzed for grain size distribution. The only other sediment data consisted of sediment surveys conducted for the Conemaugh River Lake in 1966 and 1982. The sediment surveys contained gradation curves for the bed deposit samples taken during the surveys, bed elevations at the surveyed cross sections, and observed specific weights of the sediment deposits.

Since no data were available on the gradation of the sediment for either the Conemaugh River or Blacklick Creek, the model had to be adjusted by trial and error to produce the approximate gradation observed in the bed of the reservoir. This effort produced results as shown in Figures 22-28. Identical inflowing gradations were used for the Conemaugh River and for Blacklick Creek since no data were available for either stream.

Once the gradation had been adjusted, the concentration of the inflowing sediment was adjusted to yield observed thalweg elevations and deposition volumes. In order to match the thalweg elevation for the period of 1953 to 1966, the observed concentration was multiplied by a factor of 5. A factor of 5 was felt to be reasonable since the sediment loading in the river has descreased significantly with time due to the reduction of mining activities in the watershed. The use of a multiplier of 5 predicted a higher volume of deposition than was estimated by the sediment survey, but reducing the concentration to match the observed volume resulted in a deposition gradation that did not match that observed in the prototype. Since the observed gradation of the reservoir bed was one of the few hard data points in the study, it was decided that the observed bed gradation was more important to match than the exact volume of deposition. In matching the observed gradation, the model would also produce the observed thalweg elevation. Conversely, when the model was adjusted to match the volume of deposition, neither the observed bed gradation nor the thalweg elevation could be matched.

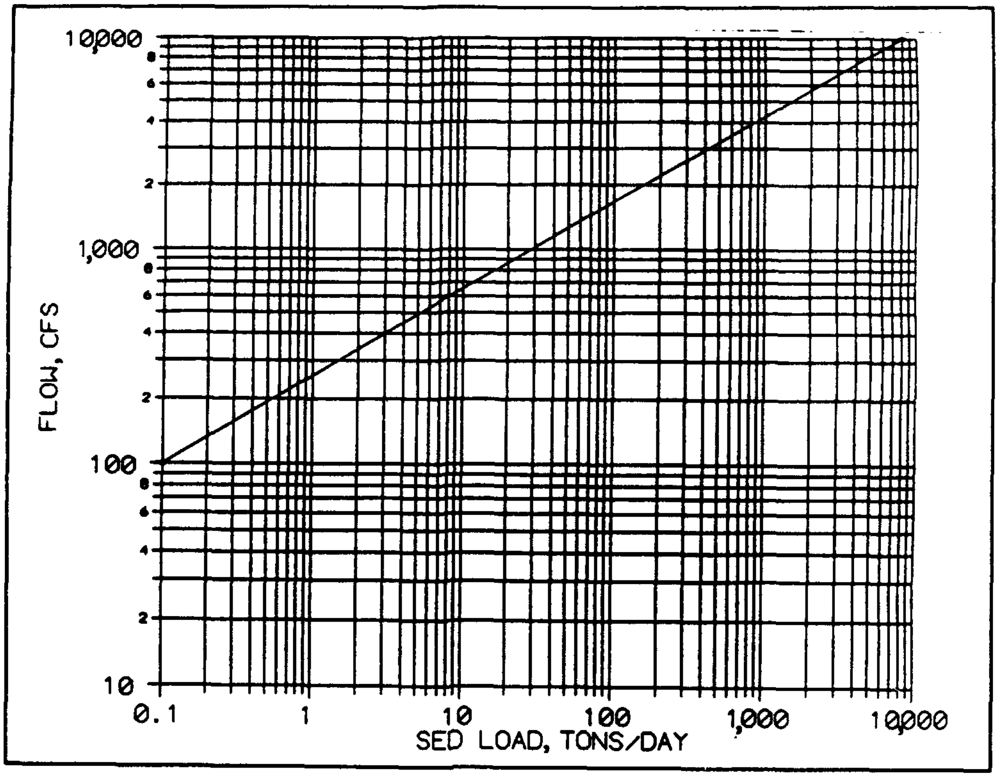

Figure 20. Blacklick sediment load versus flow curve

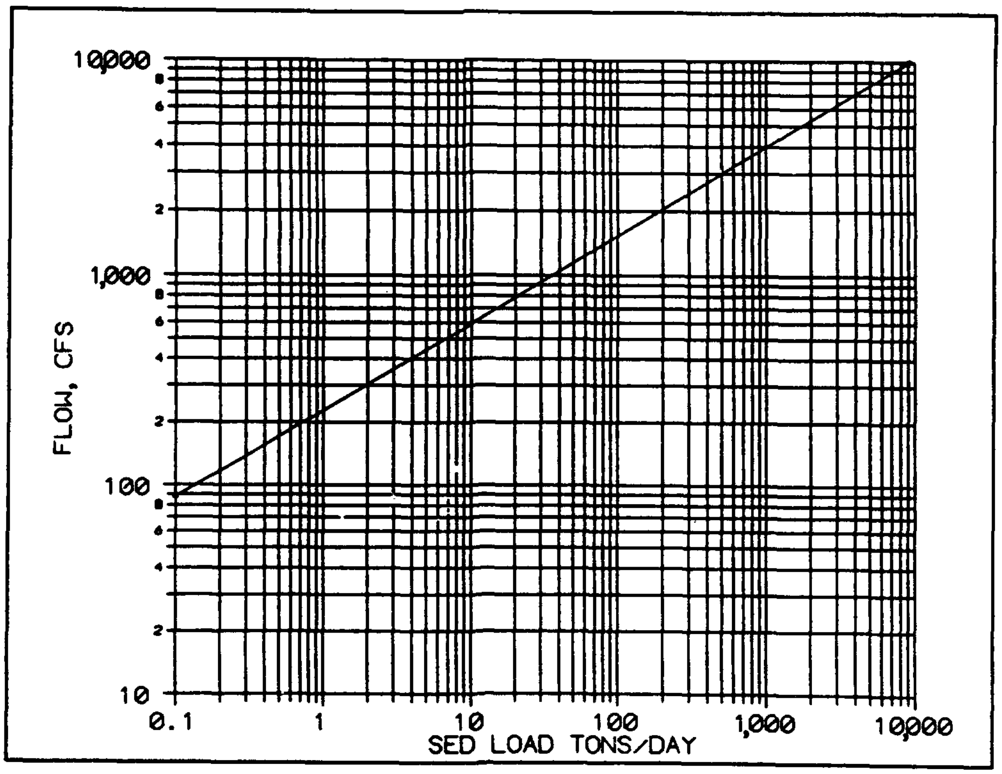

Figure 21. Conemaugh River sediment load versus flow curve

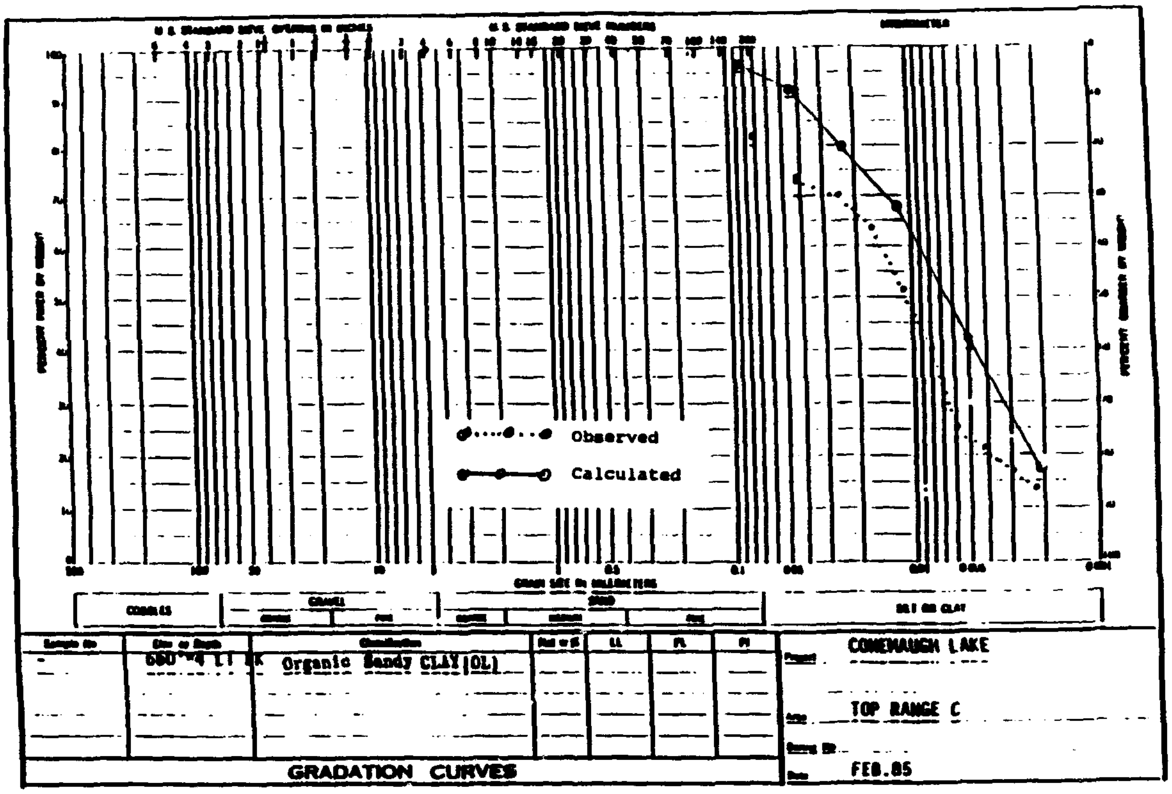

Figure 22. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range C

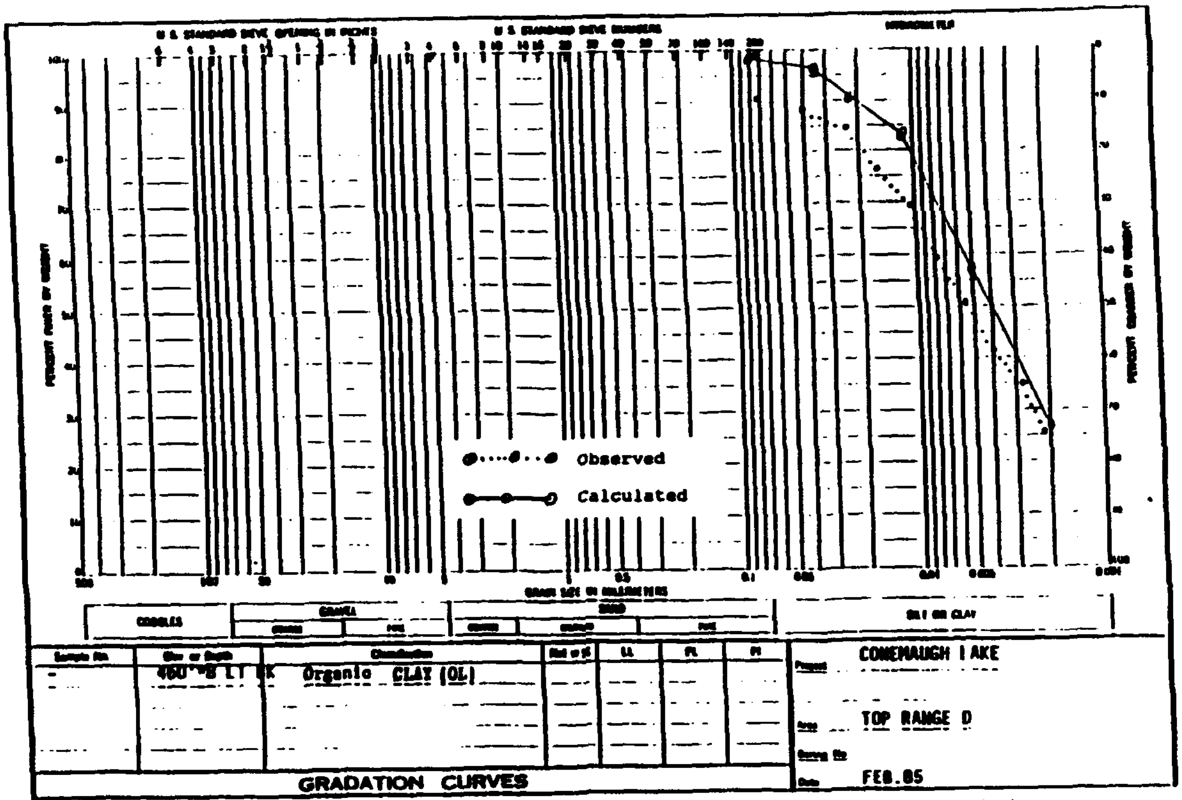

Figure 23. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range D

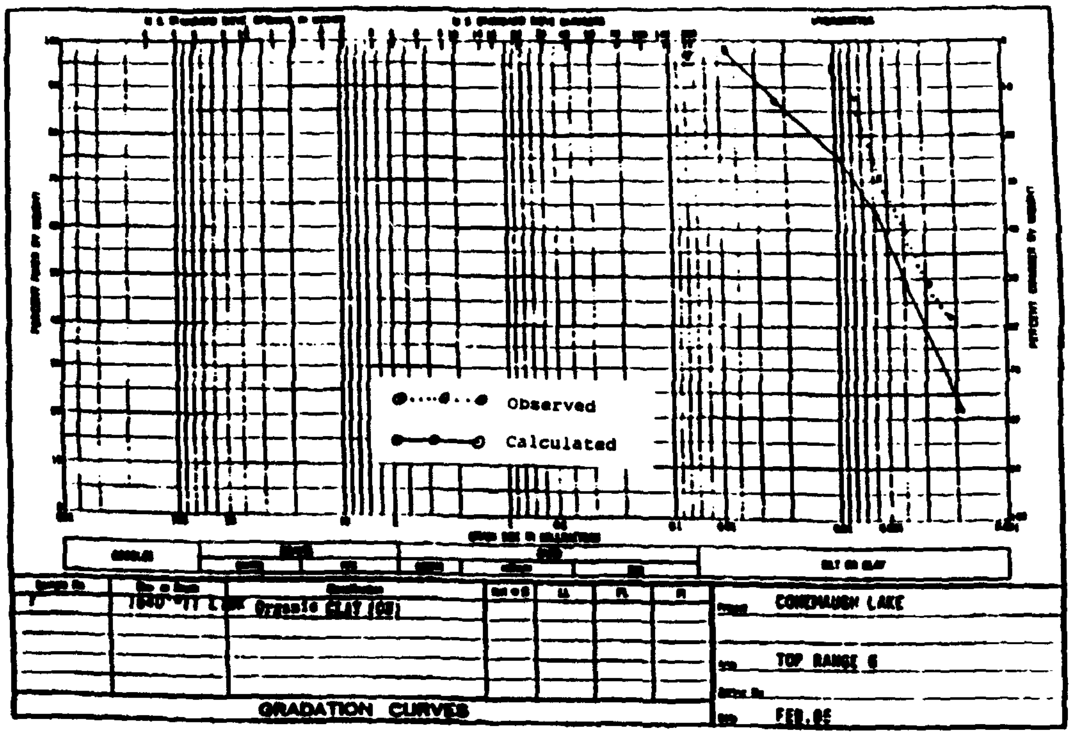

Figure 24. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range G

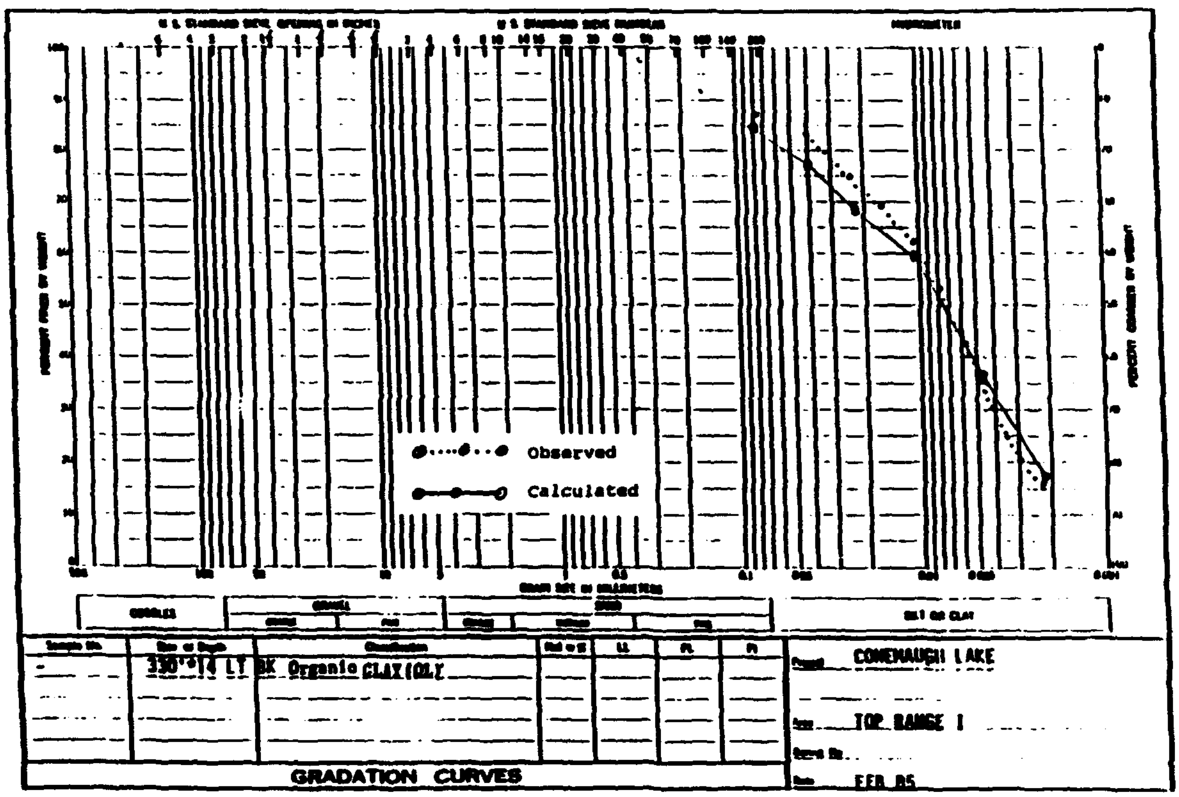

Figure 25. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range I

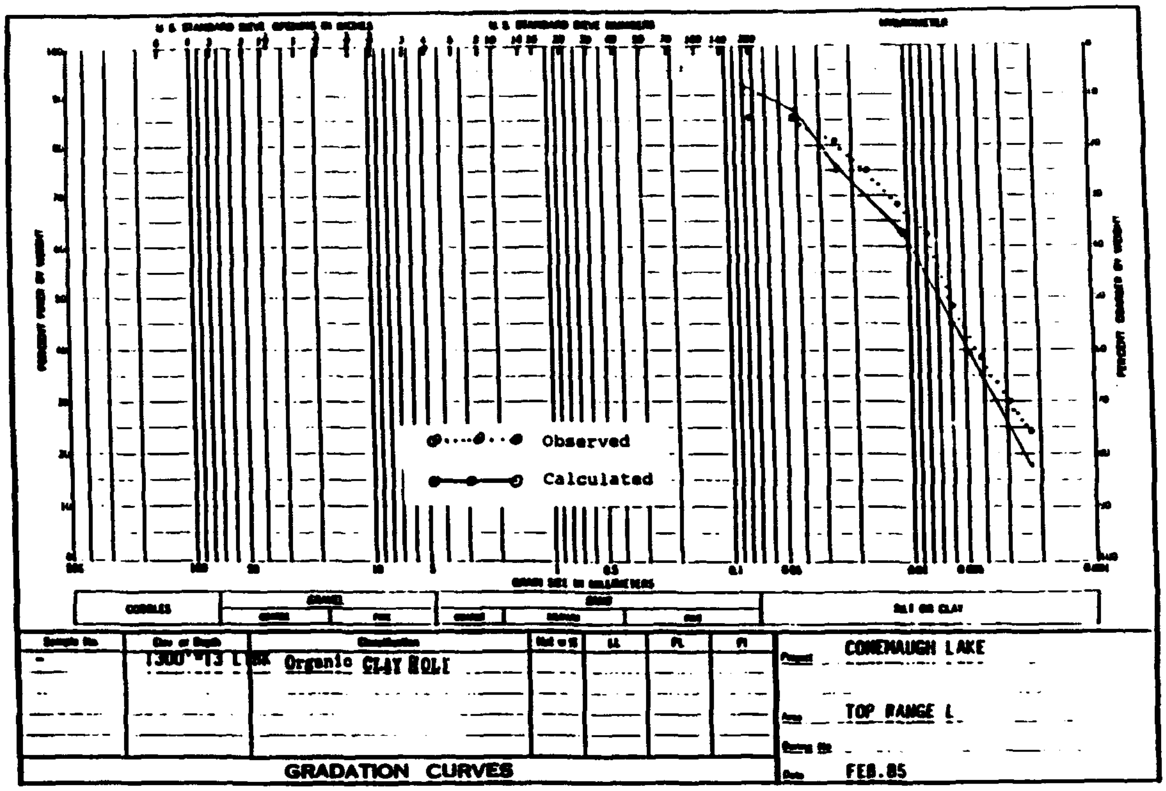

Figure 26. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range L

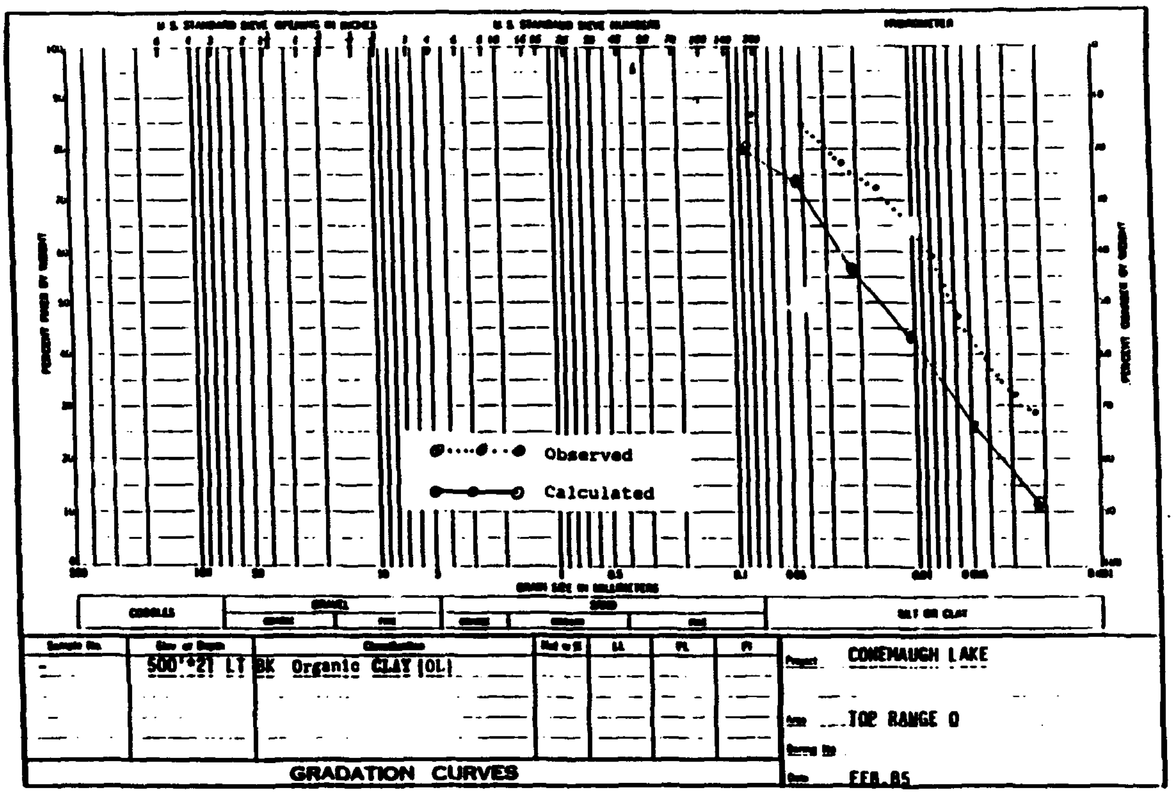

Figure 27. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range O

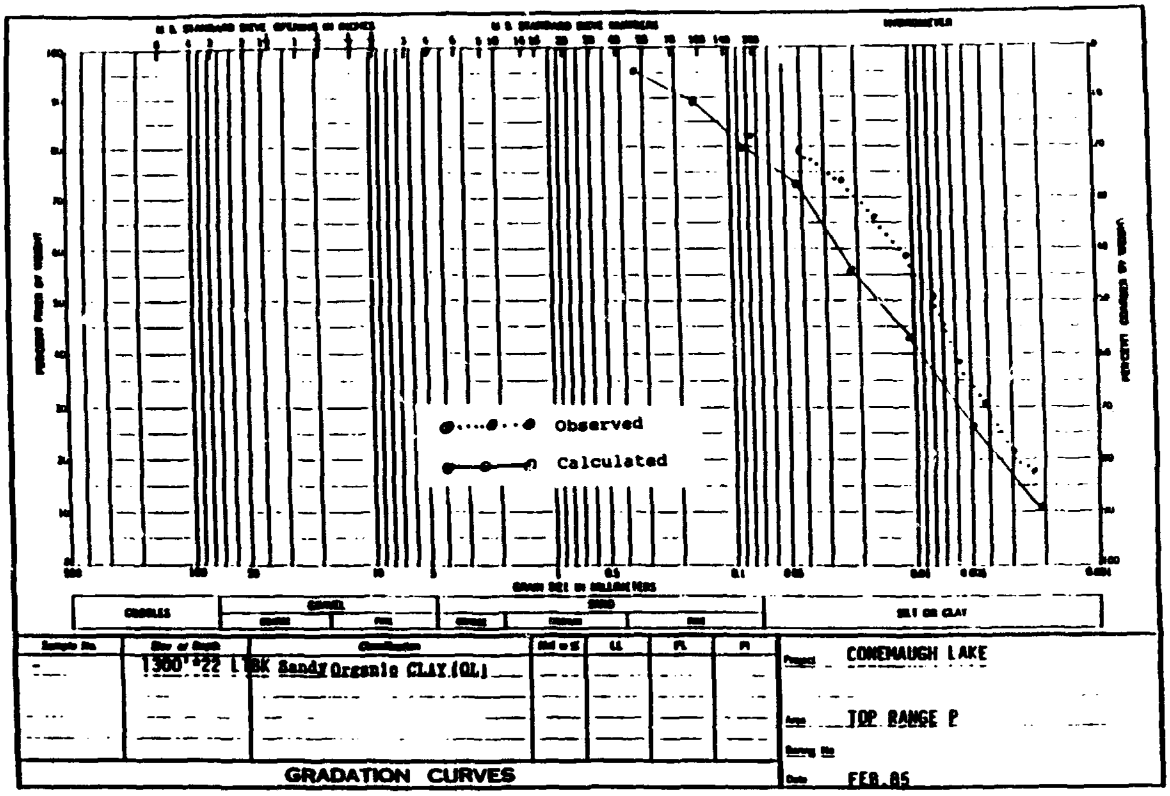

Figure 28. Comparison of observed versus TABS-1 calculated sediment gradation curves, Range P

Using the model run that best matched the observed bed gradation and thalweg elevations, the final calculated deposition for the period from 1953 to 1966 (using a specific weight of 50 lb/cu ft) was 11,342 acre-ft as compared with 5,223 acre-ft in the reservoir. The comparison of observed and calculated thalweg elevations is shown in Figure 29. After numerous runs and analyses of resulting bed profiles and grain size distributions, it was determined that the above results were the best approximation to actual conditions in the reservoir in 1966.

The TABS-1 model was then run for the period from 1966 to 1982, and concentrations were adjusted to obtain the 1982 observed bed. The best results were obtained at a concentration of 2.6 times the 1990 observed concentration. The resulting thalweg elevations are also shown in Figure 29. The volume of deposition for this run closely matched the observed volume with the model predicting 6,017 acre-ft compared with 6,119 acre-ft observed in the reservoir from 1966 to 1982.

Total volume of deposits for the simulation was 17,359 acre-ft compared with an observed value of 11,342.

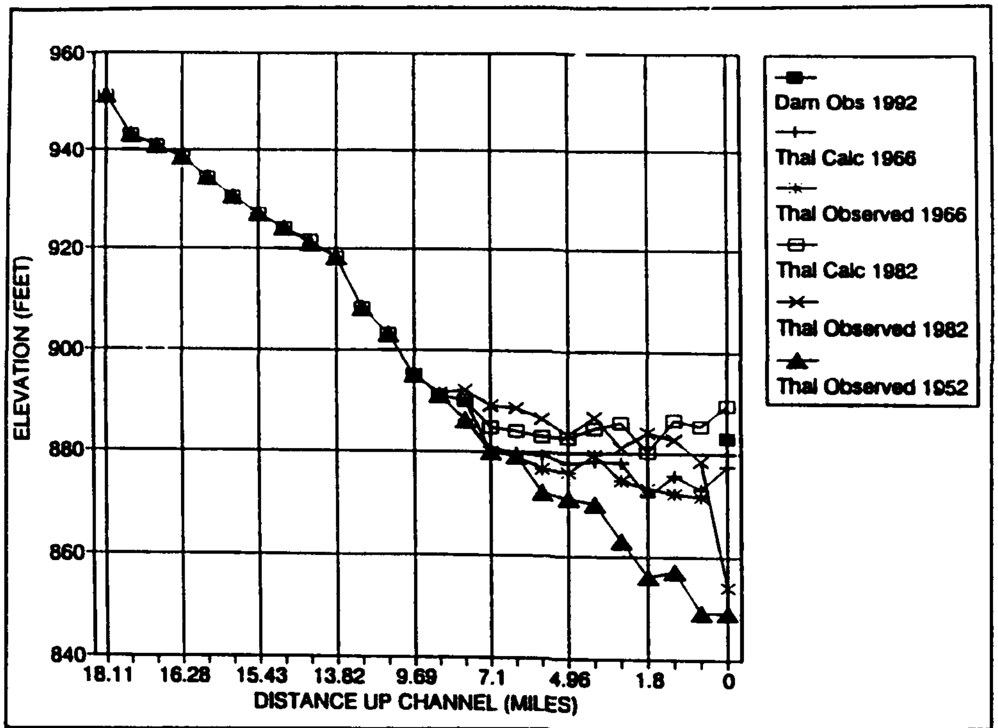

Figure 29. 1966 and 1982 thalweg elevations, observed versus calculated

Results of sediment samples taken in 1991 and analyzed at WES indicated a sediment unit weight on the order of 78 lb/cu ft. This value was used in the simulation from 1966 to 1982 and for all alternative evaluations.

Results

After model adjustment was completed, four alternatives were tested: (a) the no action or Base Test condition, (b) Plan 1, which consisted of increasing the minimum flow at the dam from 30 to 200 cfs, (c) Plan 2, which featured a dredged channel, and (d) a variation of Plan 2, which featured the dredged channel but no hydropower. All alternatives used the same reservoir operating rule curves. When the dam went into operation in 1952, the target pool elevation was 880 ft NGVD; however, the minimum pool elevation was 860 ft NGVD. Operation records indicate that the minimum pool elevation was gradually raised from 880 to 890 ft NGVD over the next 30 years. The hydropower plant began operating in February 1989, and the minimum pool elevation was raised to 900 ft at that time.

Base Test

The geometric model was changed from the 1952 adjustment geometry to the 1982 surveyed cross sections. The sediment deposits were allocated to the bed sediment reservoir and were partitioned into size classes using the gradation data collected in 1982. The observed inflowing sediment concentrations from 1991 and the particle size distributions that were developed during model adjustment were used for the inflowing sediment load. These were constant for the entire simulation hydrograph. The hydraulic roughness values were not changed from those used in the model adjustment.

The hydrologic data set was modified to pass flow through the hydropower plant and to raise the operating pool elevation to 900 ft NGVD. The direct approach for such a change was to rerun UNET. However, the UNET results developed for model adjustment contained negative flows because of numerical instabilities in the calculations. Consequently, the flow hydrographs that were used in model adjustment were modified analytically for the Base Test conditions as follows:

a. The operation of the hydropower plant was simulated by subtracting hydropower discharges from the reservoir flows at cross section 1.8, the location of the hydropower plant.

b. The change in minimum operating pool elevation was accomplished by adding the volume of water stored between elevations 880 and 900 to each day of the simulation. This procedure raised the starting elevation 20 ft to correspond to the elevation 900 pool. However, at maximum pool, the increase in elevation was only 0.5 ft.

Results from the Base Test simulation are labeled the "No Action" condition in Figure 30. Specific conditions are as follows:

a. Initial geometry adjusted to 1982 conditions.

b. Minimum pool at elevation 900.

c. Hydropower generation at mile 1.8.

d. Minimum flow bypassing the hydropower plant is 30 cfs.

The objective of this test is to predict the volume and location of sediment deposits in the reservoir for the elevation 900 minimum pool with hydropower—that is, the existing operating condition. The simulation predicted no significant new deposition upstream from the hydropower plant when compared with the 1982 reservoir delta profile, Figure 30. However, downstream from the hydropower plant, an additional 20 ft of deposition is predicted during the 30-year simulation. The volume of deposition is 9,694 acre-ft.

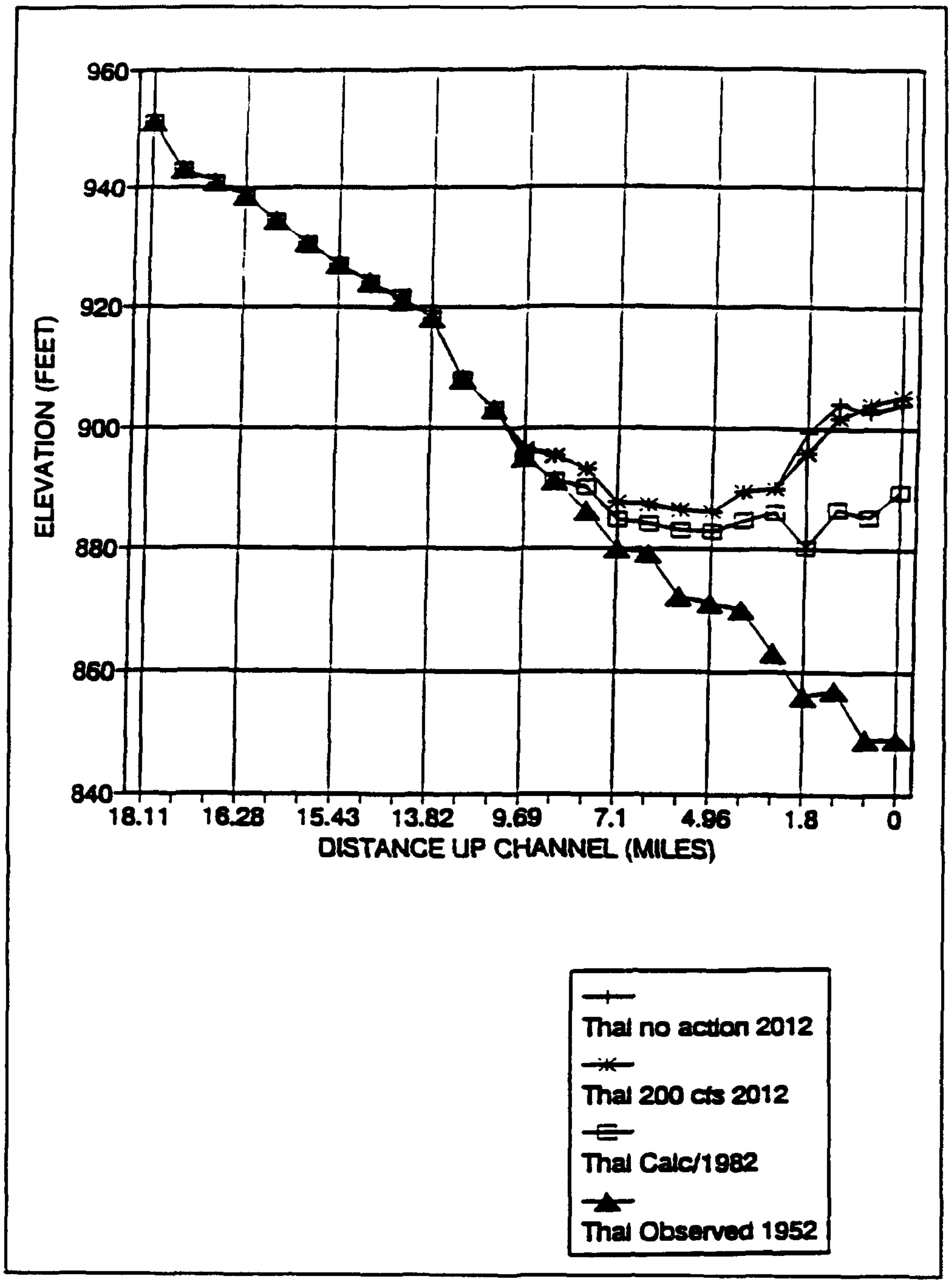

Figure 30. Calculated thalweg elevations for Base Test and Plan 1

Plan 1

This plan differs from the Base Test only in the minimum flow allowed to bypass the hydroplant. In the Base Test, that minimum was 30 cfs. In this plan, the minimum flow was increased to 200 cfs. Model conditions are as follows:

a. Initial geometry adjusted to 1982 conditions.

b. Minimum pool at elevation 900.

c. Hydropower generation at mile 1.8.

d. Minimum flow bypassing the hydropower plant is 200 cfs.

Results are shown in Figure 30. There would be about a 6-percent reduction in the volume of deposition with this alternative as compared with the Base Test. That reduction occurred between the dam and the hydropower plant.

Plan 2

This is a dredging plan supplied by the Pittsburgh District. It has a 500-ft base width at the dam (cross section 0.10) and tapers to a triangular channel at mile 0.17 upstream of the dam. The triangular channel continues to the cross section at 8.77 miles upstream from the dam. This is just downstream from the confluence of Blacklick Creek and the Conemaugh River. The hydrology was developed using the same technique described for Plan 1. However, the minimum flow bypassing the hydropower plant was set to 30 cfs, which is the same as that used in the Base Test. The initial dredged profile and the calculated bed profile after the 30-year flow simulation are shown in Figure 31. The 30-year calculated profile is labeled "Dredge+High Pool WP," which stands for the following:

a. Initial dredging.

b. Minimum pool at elevation 900.

c. Hydropower generation.

d. Minimum flow bypassing the hydropower plant is 30 cfs.

The predicted volume of deposition is 10,869 acre-ft, which is 12 percent greater than the Base Test value. That reflects the increase in the cross-sectional area as the result of dredging. The bed profile plot shows that a substantial portion of that deposition will occur downstream from mile 1.8, the location of the hydropower plant.

Sensitivity testing

Upon completion of Plan 2, the Pittsburgh District requested the evaluation of an alternative that would separate the influence of dredging from that of hydropower generation and the higher minimum pool elevation on deposition. The test combined the dredging plan with the initial reservoir operation plan. This new alternative is called Plan 2, Run 2, and it contains the following features:

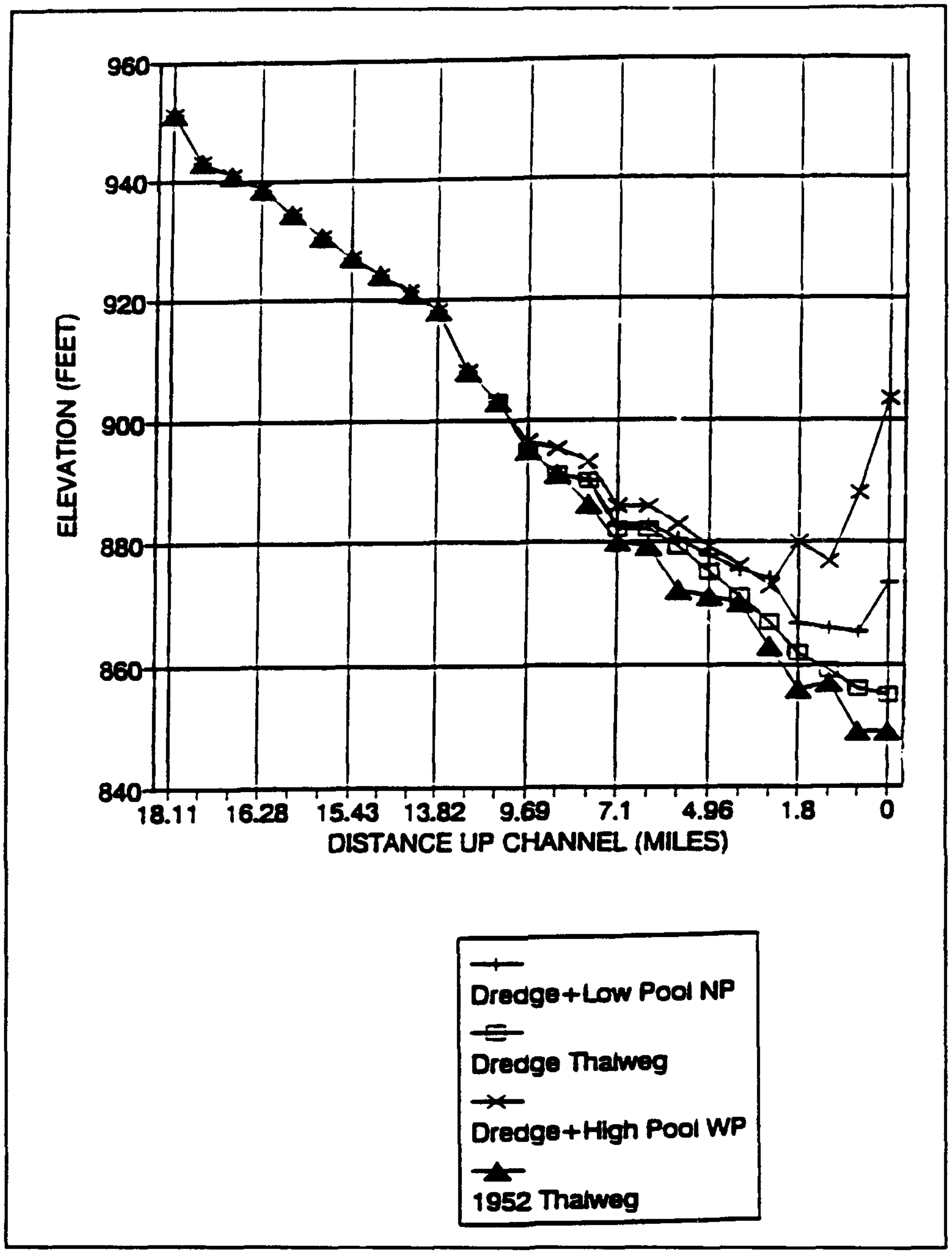

Figure 31. Calculated thalweg elevations for Plan 2

a. Initial dredging same as Plan 2.

b. Minimum pool at elevation 880.

c. No hydropower generation, same as Base Test.

d. All flow bypassing hydropower plant.

The results from this test are labeled "Dredge+Low Pool NP" in Figure 31. The volume of deposition is predicted to be 5,240 acre-ft, which is 48 percent of the Plan 2 test with hydropower.

After initial dredging to remove the sediments from the dam, it is possible that a system such as that used by Hotchkiss (1992) would be capable of removing accumulated sediments from the base of the reservoir upstream from the dam. The method used by Hotchkiss consists of an outlet pipe that is built into an existing low-level sluice gate as shown in Figure 32. The upstream end of the pipe is then extended sufficiently to reach an area of sediment accumulation, and the pipe inlet is either left on the bottom of the reservoir to continuously pass sediment or positioned by a barge such that the pipe inlet can be moved from one area of sediment deposition to another. This movement of the pipe within the reservoir can remove deposited sediment from a much wider area of the reservoir near the dam.

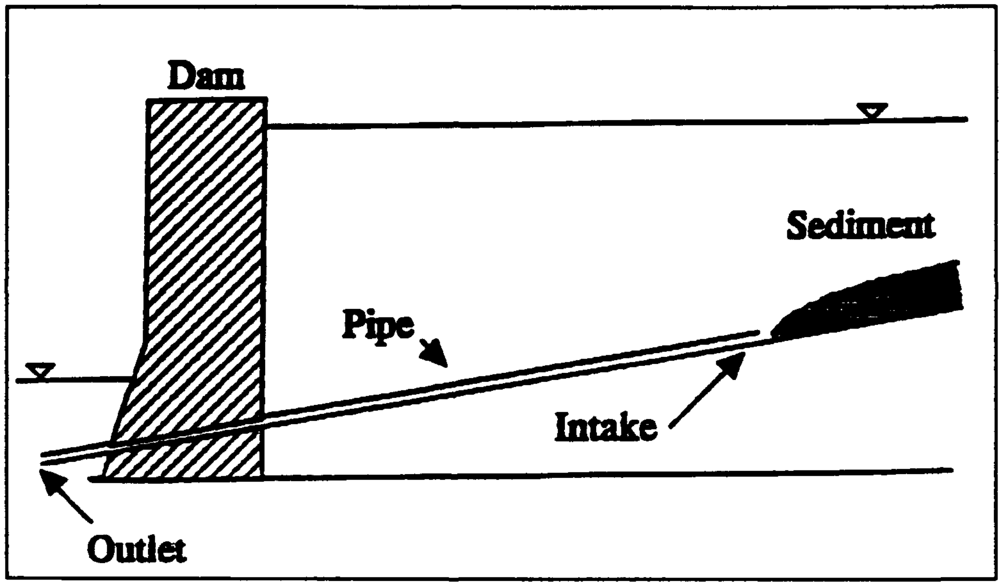

Figure 32. Single submerged pipeline release system

The downstream end of the pipe can be extended to deposit sediment at any desired location — in this case, probably into the discharge from the hydropower turbines. This will help reduce two problems. First, the water from the power turbines is out of equilibrium with its sediment load, having just deposited nearly its entire sediment load in the reservoir. This sediment deficiency must be made up with material from the bed and banks of the downstream channel. The sediment deposited upstream in the reservoir by the clear water released through the power turbines obviously causes problems in the long-term operation of the dam and needs to be removed to ensure long-term operability of the reservoir.

Since energy requirements are reportedly extremely low or nonexistent for the method used by Hotchkiss and consist almost exclusively of energy to move the barge supporting the pipe inlet, the cost of this sediment removal method should be very low after the initial equipment is in place. The rate of sediment removal from the reservoir can be controlled by means of a valve on the pipe if necessary. Within the constraints of hydraulics, the method could provide a means of removing sediment accumulations in the lower end of the reservoir at low cost.

The sediment placed in the downstream flow can be controlled to match hydropower releases or allowed to accumulate until flow was sufficient to carry the removed sediments downstream. The preferred method would probably be to match the flow of sediment from the reservoir to the flow from the hydropower plant to restore some of the deposited sediment to the downstream flow. The concentration of suspended sediment in the downstream flows should be low enough that little if any difference could be observed at the city water intakes located farther down the Conemaugh River.

Hotchkiss noted that the legal implications of this method are as yet unclear. One of the unanswered questions includes a determination of whether the discharged sediment be considered dredge material or whether it will be considered part of the natural system. There are also a number of other questions remaining to be answered. For this particular reservoir, the benefits in the reduction or control of long-term deposition in the channel should be significant enough to justify further investigation of this method. A research trial of this method could be conducted prior to full-scale implication to test the applicability of the method to the Conemaugh River Reservoir.

The agitation of sediment to facilitate its passage through the gate structure could be performed in conjunction with the method being investigated by Hotchkiss as described above. Concerns about water quality for downstream deliveries would have to be addressed if large quantities were to be released.

Sediment has been removed by the flushing of reservoirs, but this is not desirable due to the extremely high sediment concentrations that are involved. These very high concentrations can kill fish and cause sedimentation in downstream inlet structures for very long distances downstream from the reservoir. The water would also not be suitable for municipal or industrial uses.

The retention and or the removal of sediment in the area of Blacklick Creek was considered not feasible due to the very fine nature of the sediment. The material is being transported as suspended load and would not readily deposit unless conditions in a sediment trap were similar to those in the reservoir near the dam, i.e., low velocities for extended periods of time.

Sediment retention at the hydropower plant would be very difficult, again due to the very fine nature of the sediment. The passing of the sediment load through the hydropower plant would damage turbine blades, even in the event the sediment could be diverted at the hydropower intake.

From the numerical model studies, it appears that sediment will accumulate to the normal level of the pool unless some action is taken, wherever that level may be.

4 Conclusions and Recommendations

Conclusions

Based on the results of the settling and elutriate tests, the following is concluded:

a. The Conemaugh River Lake sediment exhibited zone settling with a settling rate of 0.101 ft/hr. The zone settling behavior of the sediment indicates that 16.0 acres minimum surface area would be required assuming an 8-in. dredge size.

b. Effluent total suspended solids concentration after 2.50 days under quiescent settling conditions is predicted at 60 mg/L. A minimum ponding surface area of 49 acres is required assuming a minimum of 2-ft ponding depth.

c. The removal of 350,000 yd^3 of dredged material requires 37 acres of initial storage and a dredged material storage depth of 6 ft.

d. The bulk chemical analyses indicate that the sediment to be dredged has elevated levels of metals and cyanide. PCBs were less than 0.002 mg/kg except PCB-1254 with average concentration of 0.074 mg/kg. 1,2,4 Trichlorobenzene was detected at 10.7 mg/kg.

e. The modified elutriate test indicates that the effluent from the CDF may also contain dissolved metals such as cadmium (0.0017 mg/L), barium (0.298 mg/L), iron (0.175 mg/L), and manganese (0.093 mg/L).

f. The modified elutriate test indicates that the total elutriate may contain particle-associated metals such as copper (0.001 mg/L), aluminum (0.042 mg/L), barium (0.332 mg/L), iron (0.228 mg/L), and manganese (0.092 mg/L).

g. The organic analytes and cyanide were less than detection limit in the modified elutriate test.

h. The modified elutriate test shows a total and a dissolved cyanide concentration of <0.005 ppm each. Possible explanations for no detection of cyanide in the elutriate test are the oxidation that occurs during the elutriate test, and the cyanide may be in a form that remains attached to the sediment.

i. The total effluent concentrations were calculated to be less than the Federal Water Quality Criteria, indicating no need to conduct an evaluation of mixing.

j. The predicted total effluent concentration of cadmium (0.0017 mg/L) was slightly higher than the fresh chronic criteria (0.0011 mg/L).

Based on the model testing results conducted in this study, the following conclusions are made.

a. With continued operation and no sediment removal, the sediment deposit can be expected to approach the operating level of the pool. This is true whether the pool is at elevation 880, elevation 900, or el 910 ft NGVD.

b. Dredging of sediment currently in the reservoir will increase the water depth and channel capacity upstream from the dam, but the elevation of sediment immediately upstream from the dam will return to depths associated with no dredging options within the 30 years simulated with the TABS-1 model.

c. The increase in pool elevation associated with the production of power also increases the maximum depth of deposition for both the dredge and no-dredge options.

d. The diversion of flows through the hydropower facility has reduced flow in the main channel, thus increasing sediment deposition in the reservoir between the hydropower inlet and the dam.

Recommendations

Based on the results of this study, it is recommended that the settling test results and the modified elutriate test results be utilized for the proper design of a CDF to store the Conemaugh River Lake dredged material. Based on results of the bulk chemistry analysis, it is recommended that an additional study be conducted to determine why cyanide concentration was high in the sediment and not detected in the modified elutriate test and to determine the source of cyanide contamination.

It is also recommended that a sediment removal system similar to that described by Hotchkiss (1982) be considered and perhaps an experimental trial be conducted in an attempt to control long-term deposition in the reach of the reservoir above the dam. This could be coupled with agitation near the pipe inlet to increase the effectiveness.

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Appendix A
Detailed Test Results

Table A1
Bulk Chemistry Analysis

Analyte	Concentration[1] Sample 1	Sample 2	Sample 3	Average
Semivolatiles				
Phenol	<3.3	<3.3	<6.4	<4.3
2-Chlorophenol	<3.3	<3.3	<6.4	<4.3
2-Nitrophenol	<3.3	<3.3	<6.4	<4.3
2,4-Dimethylphenol	<3.3	<3.3	<6.4	<4.3
2,4-Dichlorophenol	<3.3	<3.3	<6.4	<4.3
4-Chloro-3-Methylphenol	<6.6	<6.6	<13	<8.7
2,4,6-Trichlorophenol	<3.3	<3.3	<6.4	<4.3
2,4-Dinitrophenol	<16	<16	<32	<21
4-Nitrophenol	<16	<16	<32	<21
2-Methyl-4,6-Dinotrophenol	<16	<16	<32	<21
Pentachlorophenol	<16	<16	<32	<21
Benzoic Acid	<16	<16	<32	<21
2-Methylphenol	<3.3	<3.3	<6.4	<4.3
4-Methylphenol	0.40J	<3.3	<6.4	<4.3
2,4,5-Trichlorophenol	<3.3	<3.3	<6.4	<4.3
Benzyl Alcohol	<6.6	<6.6	<13	<8.7
N-Nitrosodimethylamine	<3.3	<3.3	<6.4	<4.3
Bis(2-Chloroisopropyl)Ether	<3.3	<3.3	<6.4	<4.3
N-Nitroso-Di-N-Propylamine	<3.3	<3.3	<6.4	<4.3
Nitrobenzene	<3.3	<3.3	<6.4	<4.3
Isophorone	<3.3	<3.3	<6.4	<4.3
Bis(2-Chloroethoxy)Methane	<3.3	<3.3	<6.4	<4.3
2,6-Dinitrotoluene	<3.3	<3.3	<6.4	<4.3
2,4-Dinitrotoluene	<3.3	<3.3	<6.4	<4.3
1,2-Diphenylhydrazine	<3.3	<3.3	<6.4	<4.3
Benzidine	<16	<16	<32	<21
3,3'Dichlorobenzidine	<3.3	<3.3	<6.4	<4.3
Bis(2-Chloroethyl)Ether	<3.3	<3.3	<6.4	<4.3
1,3-Dichlorobenzene	<3.3	<3.3	<6.4	<4.3
1,4-Dichlorobenzene	<3.3	<3.3	<6.4	<4.3
1,2-Dichlorobenzene	<3.3	<3.3	<6.4	<4.3
Hexachloroethane	<3.3	<3.3	<6.4	<4.3
1,2,4-Trichlorobenzene	12	10.5	9.7	10.7
Naphthalene	<3.3	<3.3	<6.4	<4.3

(Sheet 1 of 4)

[1] Parameter concentration units are in mg/kg.

Table A1 (Continued)				
	Concentration[1]			
Analyte	Sample 1	Sample 2	Sample 3	Average
Semivolatiles (Continued)				
Hexachlorobutadiene	<3.3	<3.3	<6.4	<4.3
Hexachlorocyclopentadiene	<3.3	<3.3	<6.4	<4.3
2-Chloronaphthalene	<3.3	<3.3	<6.4	<4.3
Acenaphthylene	<3.3	<3.3	<6.4	<4.3
Dimethyl Phthalate	<3.3	<3.3	<6.4	<4.3
Acenaphthene	0.3J	<3.3	<6.4	<4.3
Fluorene	0.89J	0.72J	1.2J	0.94J
Diethyl Phthalate	<3.3	<3.3	<6.4	<4.3
4-Chlorophenyl Phenyl Ether	<3.3	<3.3	<6.4	<4.3
N-Nitrosodiphenyl Amine	<3.3	<3.3	<6.4	<4.3
4-Bromophenyl Ether	<3.3	<3.3	<6.4	<4.3
Hexachlorobenzene	<3.3	<3.3	<6.4	<4.3
Phenanthrene	3.1J	2.6J	9.2	5.0
Anthracene	1.2J	1.04J	5.0J	2.4J
Dibutylphthalate	<3.3	<3.3	<6.4	<4.3
Fluoranthene	3.1J	2.6J	9.3	5.0
Pyrene	3.5	3.6J	13	6.7
Butylbenzylphthalate	3.3	<3.3	<6.4	<4.3
Chrysene	2.5J	2.4J	7.6	4.2
Benzo(a)Anthracene	1.4J	1.3J	5.1J	2.6J
Bis(2-Ethylhexyl)Phthalate	1.0J	<3.3	<6.4	<4.3
Di-N-Octylphthalate	<3.3	<3.3	<6.4	<4.3
Benzo(b)Fluoranthene	1.4J	1.2J	3.9J	2.2J
Benzo(k)Fluoranthene	1.4J	1.2J	3.9J	2.2J
Benzo(a)Pyrene	0.85J	0.43J	2.0J	1.1J
Indeno(1,2,3-C,D)Pyrene	<3.3	<3.3	<6.4	<4.3
Dibenzo(A,H)Anthracene	<3.3	<3.3	<6.4	<4.3
Benzo(G,H,I)Perylene	<3.3	<3.3	<6.4	<4.3
Aniline	<6.6	<6.6	<13	<8.7
4-Chloroaniline	<6.6	<6.6	<13	<8.7
Dibenzofuran	0.78J	0.74J	0.83J	0.78J
2-Methylnaphthalene	1.5J	1.4J	1.3J	1.4J
2-Nitroaniline	<16	<16	<32	<21
3-Nitroaniline	<16	<16	<32	<21
4-Nitroaniline	<16	<16	<32	<21
Total Organic Carbon	27,337	25,599	29,680	27,538
				(Sheet 2 of 4)

Table A1 (Continued)				
	Concentration[1]			
Analyte	**Sample 1**	**Sample 2**	**Sample 3**	**Average**
Metals				
Antimony	<0.005	<0.005	<0.005	<0.005
Arsenic	50	52	51	51
Beryllium	9.0	9.0	8.9	9.0
Cadmium	4.14	4.20	2.40	3.58
Chromium	49.2	52.2	45.6	49.0
Copper	82.7	84.7	82.4	83.3
Lead	587	546	577	[illegible]
Mercury	1.0	1.0	1.0	1.0
Nickel	32.2	34.2	32.4	32.9
Selenium	1.30	1.30	1.40	1.33
Silver	0.500	0.400	0.400	0.433
Thallium	2.90	3.00	2.70	2.87
Zinc	556	585	549	563
Aluminum	21,950	22,300	21,700	21,983
Barium	349	351	344	348
Iron	121,500	123,000	122,000	122,167
Manganese	960	977	986	974
Cyanide				
Cyanide	529	585	762	625
Pesticides/PCBs				
Aldrin	<0.0002	<0.0002	<0.0002	<0.0002
A-BHC	<0.0002	<0.0002	<0.0002	<0.0002
B-BHC	<0.0002	<0.0002	<0.0002	<0.0002
G-BHC	<0.0002	<0.0002	<0.0002	<0.0002
D-BHC	<0.0002	<0.0002	<0.0002	<0.0002
PPDDD	<0.0002	<0.0002	<0.0002	<0.0002
PPDDE	<0.0002	<0.0002	<0.0002	<0.0002
PPDDT	<0.0002	<0.0002	<0.0002	<0.0002
Heptachlor	0.0068	0.0078	<0.0002	0.0073
Dieldrin	<0.0002	<0.0002	<0.0002	<0.0002
A-Endosulfan	<0.0002	<0.0002	<0.0002	<0.0002
B-Endosulfan	<0.0002	<0.0002	<0.0002	<0.0002
Endosulfan sulfate	<0.0002	<0.0002	<0.0002	<0.0002
Endrin	<0.0002	<0.0002	<0.0002	<0.0002
Endrin Aldehyde	<0.0002	<0.0002	<0.0002	<0.0002
Heptachlor Epoxide	<0.0002	<0.0002	<0.0002	<0.0002
Methoxychlor	<0.0002	<0.0002	<0.0002	<0.0002
				(Sheet 3 of 4)

Table A1 (Concluded)				
	Concentration[1]			
Analyte	Sample 1	Sample 2	Sample 3	Average
Pesticides/PCBs (Continued)				
Chlordane	<0.002	<0.002	0.002	<0.002
Toxaphene	<0.002	<0.002	<0.002	<0.002
PCB-1016	<0.002	<0.002	<0.002	<0.002
PCB-1221	<0.002	<0.002	<0.002	<0.002
PCB-1232	<0.002	<0.002	<0.002	<0.002
PCB-1242	<0.002	<0.002	<0.002	<0.002
PCB-1248	<0.002	<0.002	<0.002	<0.002
PCB-1254	0.060	0.093	0.069	0.074
PCB-1260	<0.002	<0.002	<0.002	<0.002
				(Sheet 4 of 4)

Table A2
Radionuclides Analysis

Analyte	Activity, pCi/g dry
Gross alpha	2.0000
Gross beta	20.8000
Gamma Scan	
^{40}K	8.4300
^{212}PB	1.0190
^{214}BI	0.7410
^{137}CS	0.3640
^{214}PB	0.7800
^{208}TL	0.3350
^{228}AC	1.0070
^{212}BI	1.1570
^{226}RA	0.7410
^{224}RA	1.0500

Table A3
Modified Elutriate Test Chemical Data

Analyte	Concentration, ppm	
	Total	**Dissolved**
Semivolatiles		
Phenol	<0.010	<0.010
2-Chlorophenol	<0.010	<0.010
2-Nitrophenol	<0.010	<0.010
2,4-Dimethylphenol	<0.010	<0.010
2,4-Dichlorophenol	<0.010	<0.010
4-Chloro-3-Methylphenol	<0.020	<0.020
2,4,6-Trichlorophenol	<0.010	<0.010
2,4-Dinitrophenol	<0.050	<0.050
4-Nitrophenol	<0.050	<0.050
2-Methyl-4,6-Dinotrophenol	<0.050	<0.050
Pentachlorophenol	<0.050	<0.050
Benzoic Acid	<0.050	<0.050
2-Methylphenol	<0.010	<0.010
4-Methylphenol	<0.010	<0.010
2,4,5-Trichlorophenol	<0.010	<0.010
Benzyl Alcohol	<0.020	0.002J
N-Nitrosodimethylamine	<0.010	<0.010
Bis(2-Chloroisopropyl)Ether	<0.010	<0.010
N-Nitroso-Di-N-Propylamine	<0.010	<0.010
Nitrobenzene	<0.010	<0.010
Isophorone	<0.010	<0.010
Bis(2-Chloroethoxy)Methane	<0.010	<0.010
2,6-Dinitrotoluene	<0.010	<0.010
2,4-Dinitrotoluene	<0.010	<0.010
1,2-Diphenylhydrazine	<0.010	<0.010
Benzidine	<0.050	<0.050
3,3'Dichlorobenzidine	<0.020	<0.020
Bis(2-Chloroethyl)Ether	<0.010	<0.010
1,3-Dichlorobenzene	<0.010	<0.010
1,4-Dichlorobenzene	<0.010	<0.010
1,2-Dichlorobenzene	<0.010	<0.010
Hexachloroethane	<0.010	<0.010
1,2,4-Trichlorobenzene	<0.010	<0.010
Naphthalene	<0.010	<0.010
Hexachlorobutadiene	<0.010	<0.010
Hexachlorocyclopentadiene	<0.010	<0.010

(Sheet 1 of 4)

Table A3 (Continued)		
	Concentration, ppm	
Analyte	**Total**	**Dissolved**
Semivolatiles (Continued)		
2-Chloronaphthalene	<0.010	<0.010
Acenaphthylene	<0.010	<0.010
Dimethyl Phthalate	0.0054J	0.0062J
Acenaphthene	<0.010	<0.010
Fluorene	<0.010	<0.010
Diethyl Phthalate	<0.010	<0.010
4-Chlorophenyl Phenyl Ether	<0.010	<0.010
N-Nitrosodiphenyl Amine	<0.010	<0.010
4-Bromophenyl Ether	<0.010	<0.010
Hexachlorobenzene	<0.010	<0.010
Phenanthrene	<0.010	<0.010
Anthracene	<0.010	<0.010
Dibutylphthalate	<0.010	<0.010
Fluoranthene	<0.010	<0.010
Pyrene	<0.010	<0.010
Butylbenzylphthalate	0.0020J	0.0032J
Chrysene	<0.010	<0.010
Benzo(a)Anthracene	<0.010	<0.010
Bis(2-Ethylhexyl)Phthalate	<0.010	<0.010
Di-N-Octylphthalate	0.0027J	0.0032J
Benzo(b)Fluoranthene	<0.010	<0.010
Benzo(k)Fluoranthene	<0.010	<0.010
Benzo(a)Pyrene	<0.010	<0.010
Indeno(1,2,3-C,D)Pyrene	<0.010	<0.010
Dibenzo(A,H)Anthracene	<0.010	<0.010
Benzo(G,H,I)Perylene	<0.010	<0.010
Aniline	<0.020	<0.020
4-Chloroaniline	<0.020	<0.020
Dibenzofuran	<0.010	<0.010
2-Methylnaphthalene	<0.010	<0.010
2-Nitroaniline	<0.050	<0.050
3-Nitroaniline	<0.050	<0.050
4-Nitroaniline	<0.050	<0.050
Total Organic Carbon	23.4	21.8
Metals		
Antimony	<0.003	<0.003
Arsenic	<0.005	<0.005
		(Sheet 2 of 4)

Table A3 (Continued)

Analyte	Concentration, ppm	
	Total	Dissolved
Metals (Continued)		
Beryllium	<0.005	<0.005
Cadmium	<0.0001	0.0017
Chromium	<0.001	<0.001
Copper	0.001	<0.001
Lead	<0.001	<0.001
Mercury	<0.0002	<0.0002
Nickel	<0.002	<0.002
Selenium	<0.003	<0.003
Silver	<0.002	<0.002
Thallium	<0.002	<0.002
Zinc	<0.030	<0.030
Aluminum	0.042	<0.030
Barium	0.332	0.298
Iron	0.228	0.175
Manganese	0.092	0.093
Cyanide		
Cyanide	<0.005	<0.005
Pesticides/PCBs		
Aldrin	<0.00001	<0.00001
A-BHC	<0.00001	<0.00001
B-BHC	<0.00001	<0.00001
G-BHC	<0.00001	<0.00001
D-BHC	<0.00001	<0.00001
PPDDD	<0.00001	<0.00001
PPDDE	<0.00001	<0.00001
PPDDT	<0.00001	<0.00001
Heptachlor	<0.00001	<0.00001
Dieldrin	<0.00001	<0.00001
A-Endosulfan	<0.00001	<0.00001
B-Endosulfan	<0.00001	<0.00001
Endosulfan sulfate	<0.00001	<0.00001
Endrin	<0.00001	<0.00001
Endrin Aldehyde	<0.00001	<0.00001
Heptachlor Epoxide	<0.00001	<0.00001
Methoxychlor	<0.00001	<0.00001
Chlordane	<0.0002	<0.0002
Toxaphene	<0.0002	<0.0002

(Sheet 3 of 4)

Table A3 (Concluded)		
	Concentration, ppm	
Analyte	**Total**	**Dissolved**
Pesticides/PCBs (Continued)		
PCB-1016	<0.0002	<0.0002
PCB-1221	<0.0002	<0.0002
PCB-1232	<0.0002	<0.0002
PCB-1242	<0.0002	<0.0002
PCB-1248	<0.0002	<0.0002
PCB-1254	<0.0002	<0.0002
PCB-1260	<0.0002	<0.0002
Total Suspended Solids	284	
		(Sheet 4 of 4)

Table A4
Composite Sediment Compression Test Data

Date	Time, hr	Time Interval hr	Time Interval days	Interface Depth, ft
1 Mar	1000	0.00		--
	1020	0.33		6.17
	1030	0.50		6.16
	1050	0.75		6.16
	1100	1.00		6.15
	1125	1.42		6.05
	1130	1.50		6.04
	1215	2.25		5.97
	1230	2.50		5.95
	1245	2.75		5.91
	1300	3.00		5.90
	1320	3.33		5.88
	1330	3.50		5.84
	1345	3.75		5.82
	1405	4.08		5.82
	1420	4.33		5.76
	1430	4.50		5.74
	1445	4.75		5.71
	1520	5.33		5.65
	1540	5.67		5.62
	1545	5.75		5.60
	1600	6.00		5.58
	1620	6.33		5.55
	1630	6.50		5.54
	1645	6.75		5.52
	1700	7.00		5.50
	1715	7.25		5.47
	1730	7.50		5.45
	1745	7.75		5.42
	1800	8.00		5.40

(Continued)

Note: The initial slurry solids concentration and initial interface depth were 120.5 g/L and 6.22 ft, respectively.

Table A4 (Concluded)				
Date	Time, hr	Time Interval hr	Time Interval days	Interface Depth, ft
	1830	8.50		5.36
	1900	9.00		5.31
	1930	9.50		5.26
	2000	10.00		5.21
	2030	10.50		5.16
	2100	11.00		5.12
	2130	11.50		5.08
	2200	12.00		5.03
2 Mar	1000	24.00	1.00	3.86
3 Mar	1000	48.00	2.00	2.82
5 Mar	1000	96.00	4.00	2.43
8 Mar	0930	167.50	6.98	2.13
9 Mar	0945	191.75	7.99	2.07
12 Mar	1105	265.08	11.05	1.96
13 Mar	1000	288.00	12.00	1.94
14 Mar	1000	312.00	13.00	1.92
15 Mar	1330	339.50	14.15	1.90
16 Mar	0830	358.50	14.94	1.90

Table A5
Composite Sediment Zone Test Data Slurry Concentration, 120.5 g/L, 1 March

Time, hr	Time Interval, hr	Interface Depth, ft
1000	0.00	--
1020	0.33	6.17
1030	0.50	6.16
1050	0.75	6.16
1100	1.00	6.15
1125	1.42	6.05
1130	1.50	6.04
1215	2.25	5.97
1230	2.50	5.95
1245	2.75	5.91
1300	3.00	5.90
1320	3.33	5.88
1330	3.50	5.84
1345	3.75	5.82
1405	4.08	5.82
1420	4.33	5.76
1430	4.50	5.74
1445	4.75	5.71
1520	5.33	5.65
1540	5.67	5.62
1545	5.75	5.60
1600	6.00	5.58
1620	6.33	5.55
1630	6.50	5.54
1645	6.75	5.52
1700	7.00	5.50
1715	7.25	5.47
1730	7.50	5.45
1745	7.75	5.42
1800	8.00	5.40
1830	8.50	5.36
1900	9.00	5.31
1930	9.50	5.26
2000	10.00	5.21
2030	10.50	5.16
2100	11.00	5.12
2130	11.50	5.08
2200	12.00	5.03

Note: The initial interface depth was 6.22 ft.

Table A6
Composite Sediment Flocculent Settling Test Total Suspended Solids, mg/L

Time, hr	Depth from Top of Settling Column, ft						
	0.22	0.72	1.22	1.72	2.22	2.72	3.22
0.0	60.3[a]	BI[b]	BI	BI	BI	BI	BI
6.0	51.4	BI	BI	BI	BI	BI	BI
8.0	38.6	50.8	BI	BI	BI	BI	BI
12.0	18.5	25.5	BI	BI	BI	BI	BI
24.0	11.4	15.9	37.6	55.6	109.2	BI	BI
48.0	--	29.1	34.2	27.7	42.6	46.8	83.9
96.0	--	14.0	16.9	17.2	10.4	12.6	13.6

Note: The slurry concentration was 120.5 g/L.
[a] Concentration at highest port used as initial supernatant concentration.
[b] Port is below interface, and no sample was collected at this time interval.

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13. ABSTRACT *(Maximum 200 words)*

The Conemaugh River Lake is a flood-control project formed by the closure of the Conemaugh River Dam in late 1952. Over a 30-year period, an accumulation of sediment resulted in a reduction in gross reservoir storage capacity of 4.14 percent.

Sediment removal to restore flexibility of operation of the conduits at the Conemaugh River Lake Dam is required. One alternative being considered is hydraulic dredging with disposal of the dredged material in an upland confined disposal facility (CDF). The conceptual design of the CDF requires an evaluation of the settling behavior and properties of the dredged material to be placed therein to estimate storage requirements and to estimate total suspended solids concentration.

The Conemaugh River Lake and its main tributaries were modeled to investigate the effectiveness of various alternative solutions to the sedimentation problem. Model verification and adjustments were performed based on reproduction of the accumulation rate over the period 1966 to 1982.

Laboratory column tests were performed on the Conemaugh River Lake sediment. The settling behavior was observed to be typical of other sediments if hydraulically dredged and placed in a CDF. The compression test data

(Continued)

14. SUBJECT TERMS

Adjustments	Flocculent	Turbidity
Compression	Numerical model	Zone
Confined disposal facility	Sedimentation	

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were used to develop the initial storage requirements. The flocculent indicated that the suspended solids will settle by gravity.

The model testing shows that dredging of sediment in the reservoir will increase the water depth and channel capacity upstream from the dam. The model testing also shows the accumulation of sediment immediately upstream from the Conemaugh River Lake Dam within 30 years with the no-action alternative.